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THE UNIVERSITY OF ALBERTA

THE TENSILE SPLITTING TEST APPLIED TO THERMAL CRACKING OF ASPHALT PAVEMENTS

by

JOHN THOMAS CHRISTISON

A THESIS

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled

THE TENSILE SPLITTING TEST APPLIED TO
THERMAL CRACKING OF ASPHALT PAVEMENTS

submitted by John Thomas Christison

in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

The purpose of this investigation was to determine the low temperature tensile properties of asphalt concrete cores by means of the tensile splitting test and to relate these properties to the occurrence of transverse cracking on three highways in the Province of Alberta. The three highways investigated had been surfaced using 150/200 and 200/300 penetration grade asphalt obtained from three different suppliers. The tensile properties of Marshall specimens composed of 200/300 penetration grade asphalts obtained from various suppliers were also determined.

A relationship was found to exist between the maximum tensile strain of the cores and the occurrence of transverse cracking. The average maximum tensile strain of cores extracted from a section of highway which had exhibited 540 cracks per mile was approximately 40×10^{-5} in/in; whereas the average maximum tensile strain of cores extracted from a section of highway which had exhibited no cracking was in the order of 120×10^{-5} in/in. The average maximum tensile strain of the Marshall specimens varied from 40×10^{-5} in/in to 67×10^{-5} in/in depending upon the source of asphalt used.

The occurrence of transverse cracking is dependent upon the low temperature tensile properties of the asphalt concrete pavement which in turn are dependent upon the asphalt source and penetration grade.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER I

INTRODUCTION

General

The occurrence of transverse cracking of flexible pavements in Canada and the Northern United States has given impetus to highway engineers in determining possible failure mechanisms and properties of the pavement structure on which this type of pavement distress is dependent. The presence of transverse cracking results in loss of pavement performance, service life and an increase in maintenance costs. Because of these factors and the apparent lack of knowledge concerning transverse cracking, a detailed laboratory and field investigation was initiated in the summer of 1963 within the framework of the Alberta Cooperative Highway Research Program (ACHRP). This thesis is a contribution to this continuing program.

Transverse cracking can be differentiated from other forms of pavement distress in that it is not associated with a thickness deficiency of the pavement. In colder climates cracks become evident after periods of sub-freezing temperatures and therefore can be attributed to thermally induced stresses. Shields (1964), has suggested possible mechanisms promoting transverse cracking which included simple contraction of the asphaltic surface course and/or shrinkage of the subgrade.

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Considering these possible mechanisms combined with climatic influences and the wide range of materials used in the construction of our modern day highways it is apparent that the problem involved in defining the causes of transverse cracking is a complex one.

Since one of the important factors associated with the occurrence of transverse cracking is considered to be the low temperature tensile stress-strain properties of the asphaltic surface course an investigation pertaining to these properties appears warranted.

Present Status of Investigations

The initial research performed under the auspices of the ACHRP consisted of relating transverse cracking frequencies to (1) such variables as soil type, age of pavement, structural details and asphalt source (2) the mechanical properties of recovered asphalt concrete cores (3) the results of consistency tests as well as absolute viscosity determinations on asphalts extracted from surface materials and (4) volume change measurements of subgrade tube samples. The results of these early investigations have been summarized by Shields and Anderson (1964).

Field observations suggested that the occurrence of transverse cracking was associated with certain asphalt sources. (Asphalt source in this context refers more specifically to the asphalt supplier rather than the actual crude source.) This observation coupled with the fact that the tensile stresses that are induced in an asphalt pavement under thermal conditions must be resisted or relieved by the asphalt

binder led to a laboratory investigation of the tensile stress-strain behaviour of thin films of asphalt over a wide range of temperatures. The asphalts investigated were extracted from two sections of the same highway, each section exhibiting different cracking frequencies and each section composed of asphalt from a different source. The results of this investigation indicated that the tensile failure strain of the asphalt extracted from a highly cracked section was less than the failure strain of the asphalt extracted from an uncracked section of highway. This result encouraged the development of a testing program by which the influence of asphalt source on the tensile strength properties of an asphalt mix could be evaluated. Asphalts from two different sources were used in the preparation of Marshall specimens. Pavements composed of asphalts supplied by these sources had shown different cracking frequencies throughout their service life. The low temperature tensile strength properties of the Marshall specimens were evaluated by means of the tensile splitting test. The findings of this study showed that at temperatures greater than 0°F the Marshall specimens composed of the asphalt which had exhibited the greater cracking frequency also exhibited the greatest tensile failure stress and the least failure strain.

Present evidence from these investigations suggests that transverse cracking is associated with asphalt source. This apparent relationship is supported mainly by field evidence and hence a study of the tensile strength characteristics of various existing surface courses in relationship to the occurrence of transverse cracking appears warranted.

Purpose of this Investigation

The prime object of this investigation was to determine the low temperature tensile strength properties of asphalt surface cores extracted from various sections of three highways in the Province of Alberta, and to relate these properties to the occurrence of transverse cracking within the sections. The highways selected for study had been included in the previously mentioned field and laboratory investigations. The highway sections were selected where both cracked and relatively intact pavements adjoined, some sections forming only parts of large areas affected.

A second objective of this investigation was to determine the influence of asphalt source and grade upon the tensile strength properties of the extracted cores and to relate these variables (source and grade) to the occurrence of transverse cracking. Of the highways investigated, three different asphalt sources and two asphalt grades were present.

The variations in the physical properties of the extracted cores may tend to mask the variations of the tensile properties of the cores which could otherwise be associated with the asphalt source and grade. As a result the tensile properties of fifteen Marshall specimens, having identical physical properties but composed of asphalts from three different sources, were also investigated.

Limitation of the Thesis

There are many limitations to this thesis, the majority of which may be attributed to the number of cores tested and the unknown mechanical properties of the individual cores.

The stress-strain characteristics of asphalt mixes are dependent upon time and temperature. Because of the limited number of cores extracted from each section of highway it was necessary to adopt one test temperature (0°F) and one rate of loading (.06 in./min.) in order to obtain average strength values for each section. The significance of these average values may be questioned because of unknown variations in the mechanical properties of the individual cores.

A further limitation of this thesis is that the occurrence of transverse cracking is assumed to be solely promoted by thermal contraction of the surface components whereas the behaviour of the subgrade may be an overriding factor.

Organization of the Thesis

CHAPTER II contains a review of literature on the visco-elastic, tensile and thermal properties, of asphalts and asphalt mixes. These properties are pertinent to any investigation which attempts to relate the tensile properties of asphalt mixes to the occurrence of transverse cracking by employing a laboratory procedure.

CHAPTER III discusses the theory and the assumption involved in the use of the tensile splitting test on asphalt concrete samples at low temperatures.

The fourth chapter contains a brief outline of the testing procedure and testing program.

CHAPTER V is devoted to the presentation of the test results. The figures and tables in this chapter present average results of numerous tests performed on asphalt concrete cores extracted from various sections of highways and laboratory prepared Marshall specimens.

The sixth chapter discusses the results presented in the previous chapter.

CHAPTER VII contains the conclusions and recommendations for further research into the causes of transverse cracking and possible revisions to the tensile splitting test.

The APPENDICES contain theoretical aspects, detailed testing procedures and typical data sheets and calculations.

CHAPTER II

LITERATURE REVIEW

Present bituminous mix design methods consider such properties as stability, density and air voids of a mixture at elevated temperatures. While it is important that these properties be considered in relation to summer climates and stationary loading conditions, flexible pavement distresses such as fatigue under repeated loading and transverse cracking has led researchers to investigate the response of asphalts and asphalt mixes under various temperatures and loading conditions. Of particular interest to the current investigation of transverse cracking is the response and the tensile strength characteristics of asphalts and asphalt mixes.

Sharan (1965), and Gillespie (1966), reviewed the tensile stress-strain properties of asphalts and asphalt mixes in their investigations relating to the occurrence of transverse cracking. The following paragraphs review a number of investigations pertaining to the response of asphalts and asphalt mixtures to various temperatures and loading conditions. A brief resume of previous investigations pertaining to the tensile and thermal characteristics of asphalts and asphalt mixes is also presented for it is intended to compare the results of these investigations to those obtained in the current study.

Viscoelastic Behaviour of Asphalts

Rader (1935), stressed the need for defining the properties of asphalts which are employed in asphalt mixtures subjected to low temperatures. Rader states,

> "No definite conclusions concerning the effects of using a low penetration asphalt should be made without investigation of the effects of origin, method of manufacture, susceptibility and softening point of the asphalt upon the physical properties of the mixture".

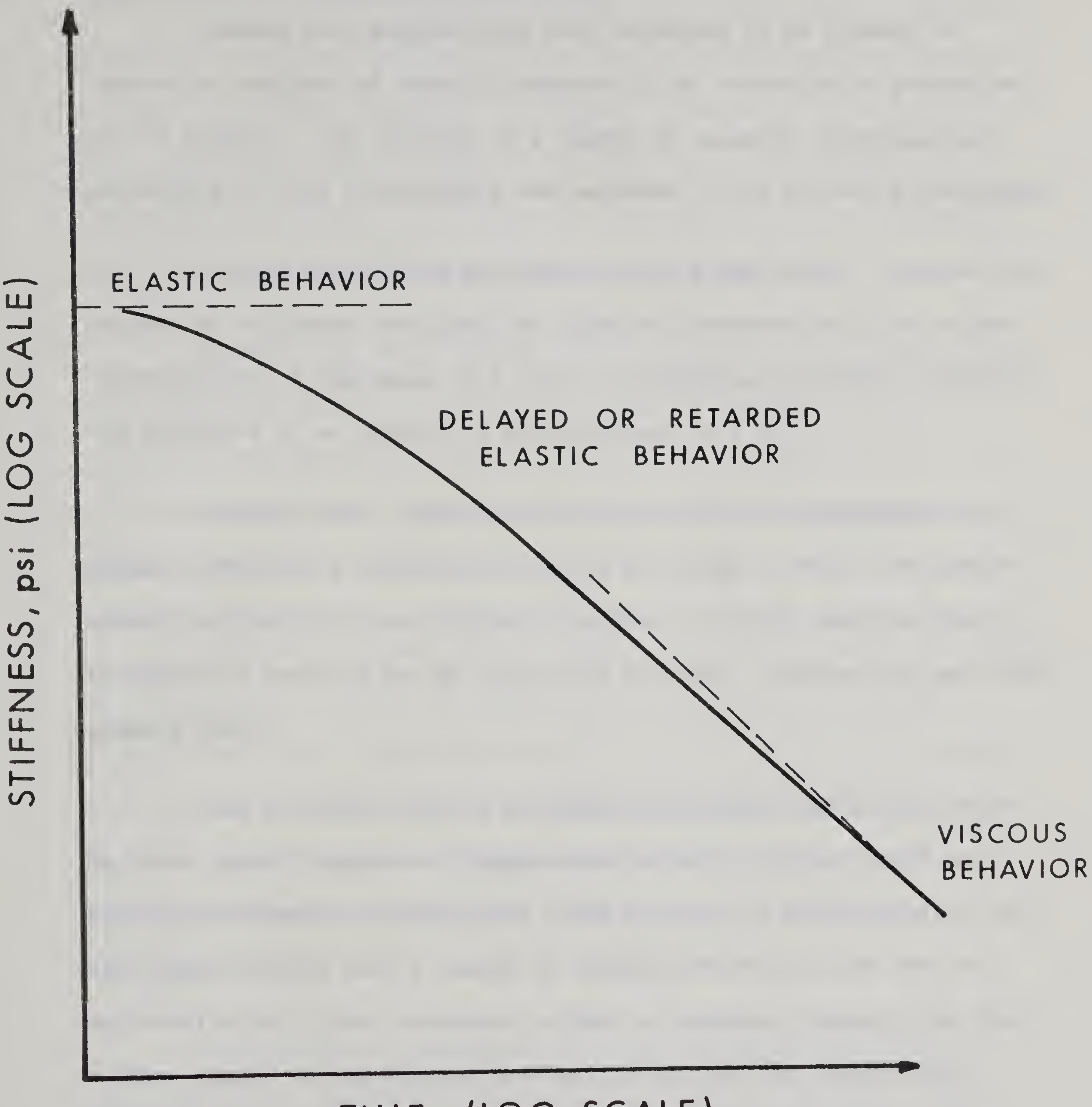
Van der Poel (1954), assessed the viscoelastic behaviour of asphalts by defining the ratio of tensile stress to tensile strain as a stiffness modulus at a particular time of loading and temperature. From constant-stress experiments (static creep tests) and dynamic tests with an alternating stress of constant amplitude and frequency on various asphalts, Van der Poel developed a nomograph from which the stiffness may be determined knowing the penetration index, ring and ball softening point, test temperature and time of loading. Van der Poel contends that the stiffness is not only a function of time and temperature but also depends upon the hardness and rheological type of the asphalt. The penetration index and the ring and ball softening point were incorporated into the construction of the nomograph as a means of measuring the hardness and rheological type respectively.

More recently Heukelom and Klomp (1964), modified the nomograph developed by Van der Poel (1954), in an attempt to obtain greater accuracy in the determination of a stiffness modulus for those asphalts having negative penetration indices (the more temperature susceptible asphalts).

FIGURE 1 illustrates the time-of-loading dependence of stiffness for a particular temperature. At very short loading times the stiffness modulus approaches the elastic modulus however as time increases a purely viscous behaviour is approached and the stiffness modulus decreases as the time-of-loading increases.

Heukelom (1966), expressed the transition from elastic behaviour to viscous behaviour in terms of three individual components of strain, an elastic strain, a delayed strain, a viscous strain and the colloidal structure of the asphalt. The chemical structure of asphalt is generally assumed to be a colloidal suspension of high molecular weight hydrocarbon molecules in low molecular weight molecules. When a load is applied to an asphalt all the hydrocarbon molecules show instantaneous deformation, the bulk effect of which can be described by the modulus of elasticity which is practically independent of time and temperature. The stress exerted on the liquid phase, the low molecular weight molecules, will cause a viscous strain to develop which is time and temperature dependent. The delayed strain is that which begins as a viscous strain but gradually approaches an elastic state. The time required to obtain elastic equilibrium is expressed as the retardation time. The retardation times of molecule agglomerates are distributed over a wide range.

Mathematical models have been developed to express the rheological behaviour of asphalts in terms of differential equations. Experimentally determined expressions, such as stiffness modulus, have been used as a means of defining the deformation properties of asphalts.



IDEALIZED STIFFNESS - TIME RELATIONSHIP

FIG. 1 (after Monismith, Secor and Secor 1965)

Viscoelastic Behaviour of Asphalt Mixes

Various test methods have been developed in an attempt to relate the response of asphalt concrete to the viscoelastic properties of the asphalt. The findings of a number of research investigations pertaining to this relationship are reviewed in the following paragraphs.

Van der Poel (1943) and Heukelom and Klomp (1964), utilized the concept of stiffness developed for asphalts, combined with the volume concentration of aggregate in a mix, in developing nomographs relating the stiffness of an asphalt to the stiffness of a mix.

Nijboer (1954), described the introduction of aggregate to an asphalt cement as a factor which limits the range in which the simple concept of elasticity and viscosity applies. Nijboer suggests that a viscoelastic analysis may be applicable for small deformations and rapid loading times.

Wood and Goetz (1956), performed unconfined compression tests on sheet asphalt samples at temperatures of 40°, 100° and 140°F and rates of deformation ranging from 0.002 in./min. to 8.65 in./min. The test data revealed that a change in temperature for any one rate of deformation has a more pronounced effect on maximum stresses than does a large change in the rate of deformation for any one temperature. Moreover, the deformation was found to take place principally in the asphalt films bounded to the aggregate and hence the temperature dependence of this behaviour was related to that of the asphalt.

From tension and compression tests at room temperature and a rate of loading of 0.12 in./min. Hargett and Johnson (1961), found that bituminous concrete surfaces can only withstand elongating deformations of about 0.3 of the deformations produced by compression. As part of a continuing study Hargett (1962), postulated that these differences in deformation properties are responsible for tension cracks developing in bituminous surfaces subjected to excessive deformation.

Monismith and Secor (1962), presented data from bending, tension and compression creep tests at 77°F on similar asphalt concrete specimens that indicate differences in tensile and compressive properties of the asphalt concrete and that the divergence in these properties is time dependent.

Davis and Krokosky (1964), concluded that the difference between the viscoelastic response of asphalt alone and that of an asphalt-aggregate composite seems to be a function of the test method employed and in particular a function of the aggregate displacement during testing. The greater the aggregate displacement, the greater the difference between the viscoelastic properties of the composite and of the asphalt.

The results of a comprehensive study performed by Monismith et al (1966), on the rheologic behaviour of asphalt concrete suggest that an asphalt concrete can be considered to be a linear viscoelastic material as long as the deformations are small (less than approximately 0.1%). The authors also contend that the stiffness concept

originally suggested by Van der Poel would serve as a useful means to incorporate the time and temperature effects within existing elastic solutions because of the present limited availability of viscoelastic solutions.

Tensile Strength of Asphalts

One of the significant factors to be considered when examining the transverse cracking of asphalt paving surfaces is the tensile strength of the asphalt surface. In the previous section the stress-strain characteristics of asphalt mixes were described as being time and temperature dependent thus it would appear that the maximum tensile strength of asphalts and asphalt mixes also depended on these parameters. This section and the following review a number of investigations pertaining to the tensile strength of asphalts and the tensile strength of asphalt mixes, respectively.

Van der Poel (1954), obtained the tensile strength of various asphalts, at temperatures ranging from -15°C to +5°C and rapid loading times, by loading strips of asphalt in tensile creep tests. The maximum tensile strength was found to be approximately 400 psi and decreasing slightly as time to rupture increased.

Mack (1957), determined the tensile force required to separate two steel plates cemented together by thin films of asphalts. Tests were conducted on asphalts from various sources and varying film thickness at 77°F and a constant rate of stress. Mack concluded that a maximum tensile strength in the order of 200 psi was developed at

optimum film thicknesses ranging from 20 to 50 microns for those asphalts investigated. Mack suggests that due to residual stresses caused by the difference in coefficients of expansion of asphalts and aggregates and stress concentrations resulting from the presence of microcavities the tensile strength of asphalt cannot be expected to exceed 200 psi.

More recently, Majidzadeh and Herrin (1965), have studied the modes of failures and the strength of asphalt films subjected to tensile stresses. The variables studied were film thickness, rate of extension, temperature and size of specimen. Tensile strength was found to decrease as the film thickness increased and approached a constant value which was dependent on the test conditions. The authors concluded that the tensile strength of thin films increases as the rate of extension increases or as temperature decreases. Three types of failure were observed, brittle fracture, tensile rupture and failure by flow. The range of film thickness at which different types of failure occurred were dependent upon the variables studied. As the rate of extension increased or temperature decreased the limits of brittle failure and flow conditions were shifted to a thicker film range.

Sharan (1965), investigated the tensile strength characteristics of thin asphalt films, the asphalt being recovered from pavements exhibiting both extensive and negligible transverse cracking. The general strength relationships were found to be similar to those reported by Majidzadeh and Herrin (1965), however it was found that at lower

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temperatures, approximately 40°F, the asphalt recovered from the cracked section had higher tensile strengths and lower unit strains to failure than the asphalt recovered from the uncracked section. Sharan suggests that the stiffness modulus of an asphalt may be used as an indication of the susceptibility of an asphalt pavement to transverse cracking.

Tensile Strength of Asphalt Mixes

Hillman (1940), performed flexure tests at a constant rate of stress on bituminous samples of varying depths and widths. The tests were conducted at -27°F, 0°F, and +27°F. The strength and stiffness of the specimens increased, as the density of the specimens was increased by added compaction and as the test temperature was decreased.

Bone et al (1954), performed tension tests on dog bone specimens and found that the tensile strength increased as the temperature decreased and reached a maximum at some temperature, usually below freezing, and then decreased at still lower temperatures.

Goetz (1955), conducted sonic tests on bituminous mixtures composed of 60-70, 85-100 and 120-150 penetration asphalt and asphalt contents of 3, 4 and 5% by weight of aggregate. The stiffness of the material was found to decrease as the temperature increased and at temperatures above 40°F the plastic component of the deformation predominated, below 30°F the elastic properties predominated.

Livneh and Shklarsky (1962), performed tensile splitting tests on laboratory prepared prismatic specimens at room temperature. Various methods of compaction were employed in the preparation of the specimens

and it was found that specimens of the same density prepared by different methods of compaction differed in strength.

Tons and Krokosky (1963), studied the tensile stress-strain patterns of a bituminous mixture at temperatures ranging from -20°F to 120°F and rates of strain varying from .004 in./min. to 0.4 in./min. The ultimate tensile stress was found to be in the order of 600 psi and occurred at a temperature of approximately 20°F. The rate of strain at low temperatures had little influence on the maximum tensile strengths. Tons and Krokosky postulated that the decrease in tensile stress, below 20°F, was due to uneven distribution of stresses within the mixture.

Domaschuk et al (1964), assessed the tensile properties of 10" x 4" x 2-1/2" specimens at rates of strain of .008 in./min. and .0008 in./min. and temperatures of +24°F, -4°F and -15°F. The ultimate tensile stress was found to vary from 400 psi at -15°F to 150 psi at 24°F. The rupture strain increased from .005 in./in. to .020 in./in. for the same temperature interval.

Heukelom and Klomp (1964), have tabulated the rupture stresses and strains of asphalt bound base materials, obtained by dynamic testing procedures at various temperatures. The maximum tensile stress reported is 1420 psi and the minimum tensile strain 1100×10^{-6} in./in.

Monismith et al (1965), have presented data from creep and constant rate of strain tension tests, performed at 40°F on a single

asphalt concrete at one asphalt content. The peak tensile strength was found to be in the order of 600 psi and the lowest value of tensile strain was found to be approximately 1000×10^{-6} in./in.

Heukelom (1966), has suggested that the fracture properties of a mix can be separated into two parameters, the stiffness of the asphalt and a "mix factor" which is independent of temperature and loading time but dependent upon the proportions of asphalt cement, grading of the materials and compaction of the mixture.

Breen and Stephens (1966), employed the tensile splitting test in evaluating the tensile strength of bituminous concrete Marshall specimens at temperatures from 0 to 40°F. Their results indicate that the ultimate strength of a gravel-sand mix, having asphalt contents ranging from 4.5 to 7.0 percent by total weight of aggregate, increased as the temperature decreased however this increase was more pronounced with a decrease in temperature from 40 to 20°F than from 20 to 0°F.

Gillespie (1966), also used the tensile splitting testing to investigate the tensile stress-strain characteristics of Marshall specimens at low temperatures. The specimens were prepared with asphalts of two different penetration grades, 150-200 and 200-300, and of asphalt contents of 5, 6 and 7 percent. The test results revealed that at temperatures greater than 0°F the specimens possessing the higher penetration grade asphalt exhibited less tensile rupture strain than the lower penetration grade specimens. At 0°F the rupture strain was approximately 140×10^{-5} in./in. for both grades of asphalts and the

ultimate tensile stress of the higher penetration grade specimens was found to vary from approximately 375 psi to 450 psi, increasing as the asphalt content increased. The corresponding figures for the lower penetration grade specimens were 340 psi and 415 psi respectively.

Thermal Properties of Asphalts and Asphalt Mixes

Although little is known of the exact mechanisms contributing to transverse cracking their occurrence has been attributed to thermal effects. The following paragraphs are limited to the findings of tests performed under the ACHRP and which are pertinent to the current investigation.

The coefficient of thermal expansion of asphalt cores extracted from various highways within the Province of Alberta have been determined; however these results do not distinguish between various sections exhibiting varying cracking frequencies. The range in coefficients* was found to be from 1.23×10^{-5} in./in./°F to 1.43×10^{-5} in./in./°F which agrees favourably with 1.30×10^{-5} in./in./°F suggested by Monismith et al (1965).

More recently the glass transition temperature of asphalts extracted from surface courses of various highways have been determined. The glass transition temperature (T_G) has been defined as that temperature at which the translatory motion of molecules has ceased because the forces binding them together are so strong that the thermal energy is

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* Data shown was obtained from the Highways Division, Research Council of Alberta.

insufficient to overcome these forces. When applied to asphalt cements the glass transition temperature may be considered a measure of the temperature susceptibility of an asphalt. An asphalt having a high T_G is more temperature susceptible than one having a low T_G because of its more brittle nature at higher temperatures.

The results of this investigation have not as yet been fully evaluated; however preliminary results indicate that the T_G for those asphalts investigated range* from approximately -10°F to +10°F and may provide an insight into variations in cracking frequencies.

Summary

The following paragraphs summarize the findings of the previously cited investigations. It is further intended to discuss and compare certain of these findings to the results of the current investigation.

The viscoelastic behaviour of an asphalt has been defined in terms of a stiffness modulus which is dependent upon time, temperature, hardness and rheological type of the asphalt. Nomographs have been developed relating the stiffness modulus to the penetration test, the ring and ball softening point test, the time of loading and the test temperature. The transition from elastic behaviour to viscous behaviour can be expressed in terms of three individual components of strain and the colloidal structure of the asphalt.

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* Data shown was obtained from the Highways Division, Research Council of Alberta.

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The nomographs developed for asphalts have been further developed in an attempt to relate the viscoelastic behaviour of an asphalt to the response of an asphalt mix under various temperatures and loading conditions.

The tensile and compressive properties of an asphalt concrete are known to differ, however investigations defining this difference at various times of loading and temperatures have been limited.

The tensile strength of asphalt films has been found to vary widely with the flow properties of the asphalt and testing parameters such as time, temperature and film thickness. At low temperatures, the temperature dependence of an asphalt mix has been related to the behaviour of the asphalt and there appears to be a relationship between the occurrence of transverse cracking of asphaltic surface courses and the low temperature tensile strength properties of asphalts extracted from these courses.

The investigations into the tensile strength characteristics of asphalt mixes have resulted in four findings which are pertinent to the current investigation. These are:

1. An increase in tensile strength as temperature decreases followed by a decrease in tensile strength with a further decrease in temperature.
2. Predominately elastic behaviour of asphalt mixes at temperatures below approximately 30°F.

3. At low temperatures the rate of loading has little effect on the ultimate tensile strength of asphalt concrete.
4. Variations in tensile strength characteristics of Marshall specimens composed of asphalts from different sources.

In the investigations cited the maximum tensile strength of asphalt concrete at low temperatures appears to be in the order of 600 psi and the minimum tensile strain in the order of 1000×10^{-6} in./in.

The tensile strength properties of asphalt mixes at low temperatures have been evaluated on laboratory prepared specimens and the majority of these investigations have been performed by employing an uniaxial loading condition. A study of the low temperature tensile properties of asphalt concrete cores, assuming a two-dimensional state of stress appears warranted.

CHAPTER III

THE TENSILE SPLITTING TEST

This chapter will review the basic assumptions and boundary conditions upon which the theoretical analysis of the tensile splitting test is developed and discusses those factors which cause laboratory results to differ from the theoretical solution. The mathematical relationships representing the stress distributions within a thin cylinder when subjected to point loading on its peripheral area are presented in Appendix A.

Theory of Elasticity

The theoretical solution of the tensile splitting test is based on the theory of elasticity. The theory assumes the material

1) to be homogeneous,
2) to obey Hooke's law, which states that a constant linear relationship exist between stress and strain,
3) to obey the law of superposition, which states that the strain in a given direction is equal to the sum of the strains produced by the individual stresses in this direction.

Heterogeneity of Asphalt Mixes

An asphalt mix is composed of asphalt, aggregate and air thus constituting a three phase system and in the presence of water a four

phase system. Although heterogeneous, the effect of the assumption of a homogeneous material on the general distribution of stress cannot be determined, although it is probably small Wright (1955).

Stress Distribution

Frocht (1948), presents the following stress distribution within a cylindrical specimen when placed between two loading surfaces and loaded along two opposite generatrices of the specimen. Along the horizontal axis the tensile and compressive stresses vary from $2P/\pi dt$ and $-6P/\pi dt$ respectively at the centre of the specimen, to zero at the circumference. Along the vertical axis (that axis coinciding with the direction of the point loads) the horizontal tensile stresses have a constant value of $2P/\pi dt$ whereas the vertical compressive stresses vary from $-6P/\pi dt$ at the centre of the specimen to infinity at the loading points.

where:

P = the magnitude of the concentrated load

d = the diameter of the specimen

t = the thickness of the specimen.

The above stress distributions were derived by assuming a state of plane stress existing in the specimen. (i.e. The influence of the intermediate principal was neglected.) This condition can only be attained in a cylinder of finite thickness and hence as the thickness of the cylinder decreases the more closely the theoretical solutions are satisfied.

The occurrence of an asphalt mix possessing plastic properties cause the stress-strain characteristics of the mix to deviate from Hooke's law. Such plastic action generally increases the load required to break the specimen by decreasing the high compressive stresses at the load points and equalizing the stress concentrations in the tensile region.

Effect of Loading Strips

The stress equations indicate that an asphalt mix must have a compressive strength at least three times its tensile strength if the specimen is to fail in tension. With a concentrated load the specimen will fail at the load points due to compression and not along the vertical axis of the specimen due to tensile stresses.

Mitchell (1961), states the following effect of replacing the concentrated load with a distributed load.

> "The distributed load will change the S_x stress from tension to compression in the vicinity of the plate. Experimental evidence indicates that this condition is sufficient to retard the compressive failure at the load points so that the cylinder fails due to tensile stresses at the centre. The tensile stress is generally unchanged over approximately 3/4 of the diameter of the specimen and the general stress equations due to a distributed load reduces in the centre to the same relationships that were derived for point loads".

As the ratio of the width of the loading strip to the diameter of the specimen decreases the more closely is the theoretical stress distribution approached. The loading strip must be made of a material as to be sufficiently pliable so as to conform to the surface irregularities of the specimen, thus eliminating stress concentrations at the bearing surface.

Law of Superposition

Assuming a plane stress condition, the strain measured across the vertical diameter of the specimen will be the result of compressive stresses in the vertical direction and tensile stresses in the horizontal direction. According to the theory of elasticity the total strain in the horizontal direction may be represented by the equation:

$$e_x = \frac{1}{E} [S_x + uS_y]$$

where:

e_x = total horizontal strain (in./in.)

E = modulus of elasticity (stiffness) (psi)

S_x = principal horizontal tensile stress ($2P/\pi dt$)

S_y = principal vertical compressive stress ($6P/\pi dt$)

u = Poisson's ratio

In order to determine the stiffness of a specimen, (the modulus of elasticity assuming no viscous response) that portion of the total horizontal strain due to the tensile stresses alone must be evaluated and hence Poisson's ratio must be known.

Poisson's Ratio

A limited number of investigations have been published that define Poisson's ratio for various asphalt mix compositions, temperatures and loading conditions. Monismith and Secor (1962), computed Poisson's ratio from volume change measurements during unconfined constant rate strain tests. The mix was composed of 3/8 inch maximum size aggregate, 85-100 penetration asphalt cement and 5.1 percent

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Asphalt content by weight of dry aggregate. The results of these tests are presented in TABLE I.

TABLE I

POISSON'S RATIO COMPUTED FROM VOLUME CHANGE MEASUREMENTS

(Constant Rate of Strain Tests)

Temperature °F	Load Rate (in./min.)	Poisson's Ratio
40	0.01	0.371
40	0.10	0.358
40	1.00	0.305[a]
77	0.01	0.492
77	0.10	0.484
77	1.00	___[a]
140	0.01	0.495
140	0.10	0.498
140	1.00	___[a]

[a] Volume change difficult to record at this load rate.
(after Monismith and Secor, 1962)

The results indicate that Poisson's ratio is temperature dependent, increasing as the temperature increases and it appears that the ratio is also dependent on the loading rate at lower temperatures, however this dependence decreases as the temperature increases.

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Assuming Poisson's ratio equal to 0.33 for the test conditions employed and substituting this value together with the principal stresses into the equation representing the total horizontal strain yields:

$$e_x = \frac{1}{E} \left[\frac{2P}{\pi dt} + (0.33)\,\frac{6P}{\pi dt}\right]$$

$$e_x = \frac{P}{\pi dtE} \quad (2 + 2)$$

Based upon the above assumption, the strain due to the tensile stress would appear to be in the order of half the measured value of the total horizontal strain.

Failure Conditions

The stress-strain diagrams for an asphalt concrete at low temperatures and/or high rates of loading generally exhibit an initial elastic range followed by a plastic range. The inelastic portion of the stress-strain diagram is controlled by the amount of plastic strain and thus the ultimate tensile stress is governed by a plastic strain phenomena. The ultimate tensile strength, although governed by a plastic strain phenomena, is generally determined by the use of elastic equations; however the strain in the plastic range is time and temperature dependent and hence a linear relationship between tensile stress (as determined from elastic equations) and tensile strain no longer exists.

In the testing procedure followed the horizontal strain across the vertical axis of the specimen was determined by the use of a Tuckerman Strain Gauge and the assumption was made that the horizontal tensile stress produced a strain equal to half the measured strain.

The error involved in this assumption increases in the plastic region because of the increase in Poisson's ratio from the elastic to the plastic zone. The magnitude of this error is neglected in elastic solutions and therefore Poisson's ratio (0.33) was assumed to be constant throughout each test.

As previously mentioned, the tensile splitting test has been used by Breen and Stephens (1966) and Gillespie (1966), to evaluate the low temperature tensile strengths of Marshall specimens. Breen and Stephens combined load measurements with corresponding vertical movements of a loading head and presented the test results in terms of work to failure. Gillespie utilized the elastic equations, representing the stress distribution within the specimens, and combined the computed tensile stresses with measured strain values, to express the failure conditions of the specimens in terms of a tensile stress-strain relationship. Both of these approaches have advantages. The work to failure approach is independent of the elastic or viscoelastic behaviour of the specimen whereas the tensile stress-strain relationship lends itself to comparisons with other tensile strength investigations. The approach used is largely dependent upon the ultimate purpose of the investigation.

Further details of the tensile splitting test are given in Appendix A.

CHAPTER IV

OUTLINE OF TESTING PROGRAM AND PROCEDURE

Selection of Test Method

The testing program involved the determination of the tensile stress-strain characteristics of asphalt concrete cores from various highways and laboratory prepared Marshall specimens. The tensile splitting test was originally developed for testing portland cement concrete cylinders; however recent investigations by Breen and Stephens (1966) and Gillespie (1966) have shown that tensile properties of asphalt concrete can also be evaluated by means of this test at temperatures below 40°F. The testing procedure and the approach used in evaluating the tensile properties of the asphalt cores and Marshall specimens in the current investigation was similar to that employed by Gillespie (1966).

Testing Program

The three highways selected for study were 21-A-1, 34-A-1 and 57-A. These highways constitute a small percentage of the main highway system in the Province of Alberta and as previously mentioned, these highways have been included in the earlier laboratory and field investigations performed within the framework of the ACHRP. As the current study is a preliminary investigation, initiated to determine if the tensile properties of asphalt cores can be associated with transverse

cracking only a limited number of cores from each highway were made available prior to the commencement of the investigation.

The highways studied presented the desirable variables of (1) variations in frequency of transverse cracking within each highway (2) variations in asphalt source and (3) differences in asphalt grade.

The tensile properties of asphalt cores extracted from three sections of highway 21-A-1 were evaluated. Transverse cracking of these three sections ranges from an average 229 cracks per mile to 0 cracks per mile. The asphalt surface course of all sections is composed of 150/200 penetration grade asphalt obtained from asphalt supplier No. 10*.

The tensile properties of cores from both the upper and lower courses of five sections of highway 34-A-1 were determined. The transverse cracking frequencies of these five sections range from an average of 540 cracks per mile to 45 cracks per mile. The initial surface course of this highway was constructed using 150/200 penetration grade asphalt obtained from supplier No. 5. After approximately two years of service the initial surface course was overlayed by the existing surface course which was constructed using 200/300 penetration grade asphalt obtained from supplier No. 1. Prior to placement of the present surface course the occurrence of transverse cracking had been negligible within all five sections.

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* Asphalt suppliers are referred to by numbers which correspond to those used by Shields and Anderson (1964).

The tensile properties of asphalt concrete cores extracted from both the upper and lower lifts of two sections of highway 57-A were evaluated. Both sections were constructed using 150/200 penetration grade asphalt. The asphalt for one section was obtained from supplier No. 1, this section contains an average 348 transverse cracks per mile. The asphalt for the second section was obtained from supplier No. 5 and this section has shown no apparent pavement distress due to transverse cracking.

The Marshall specimens were prepared as replicates of the mixture design recommended for the construction of three sections of highway in south-central Alberta. The proposed mixture design is contained in Appendix D. The Marshall specimens tested were composed of one aggregate gradation and a single asphalt content, (5.5%) however the asphalts were obtained from three different suppliers, (Nos. 1, 2 and 3) and therefore the specimens served as a means of evaluating the influence of asphalt source on the low temperature tensile properties of asphalt concrete mixes.

Loading Apparatus

General views of the loading apparatus and a sample during testing and immediately after failure of the sample are shown in FIGURE 2 and FIGURE 3 respectively.

A 5 ton capacity gear driven, Wykeham Farrance Model 57, compression machine was employed as the loading device. This machine can be operated at speeds ranging from .075 in/min to .00045 in/min.

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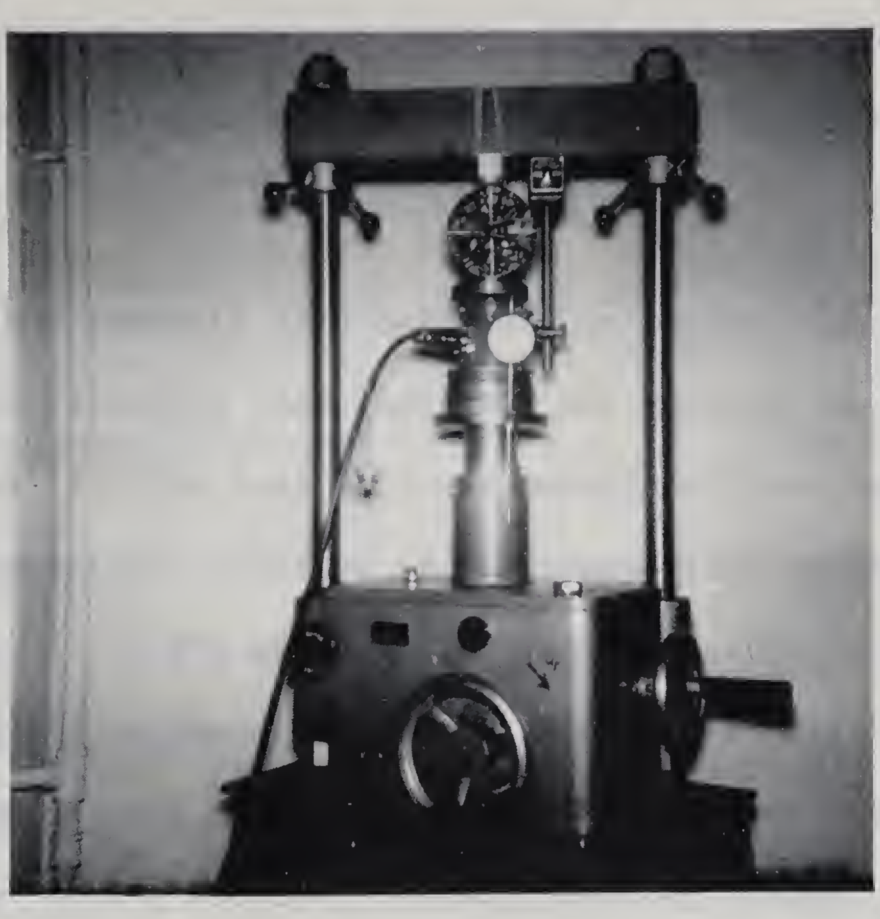

FIG. 2 LOADING APPARATUS AND SAMPLE DURING TESTING

FIG. 3 LOADING APPARATUS AND FRACTURED SAMPLE

The limited number of cores extracted from each section of highway restricted the investigation to one rate of loading, which was adopted as .06 in/min.

A 5 ton capacity load cell used in conjunction with a battery operated self-balancing load indicator was employed as the load measuring device. The load cell was positioned on a spacer plate which rested on the loading platform of the compression machine. A circular steel plate, 1/2" x 3", was designed and fabricated so as to rest firmly, in a horizontal position, on the top of the load cell. The asphalt concrete samples were placed between this plate and the upper cross frame of the loading machine. Plywood loading strips, 3" x 1/2" x 1/4", were placed at the top and bottom of the vertical axes of the samples. Plywood loading strips were used because of the materials ability to conform to any surface irregularities of the samples. Preliminary investigations revealed that a loading strip 1/2 inch wide was the minimum width which could be used without causing eccentric movement of the samples during loading.

Further details of the loading apparatus and the calibration of the load cell are contained in Appendix B.

Rate of Loading

Although the compression machine was set at a rate of loading of .06 in/min, slackness in the component parts of the machine resulted in a lower rate of loading. In order to determine the actual rate of loading for each test a strain dial having a sensitivity of 10^{-4} inches

was magnetically mounted to the upper cross frame of the compression machine with the stem of the dial resting on the loading platform of the machine. Dial readings were taken at known times during each test. The dial was removed from its position prior to failure of the samples because the sudden and brittle failure modes of the samples would have resulted in damage to the dial.

Strain Measurement

As previously mentioned the horizontal strain across the vertical axes of the samples was determined by the use of a Tuckerman Optical Strain Gauge. The Tuckerman Strain Gauge is capable of measuring strains as small as 2 micro inches per inch. Gillespie (1966), utilized the Tuckerman Strain Gauge in conjunction with the tensile splitting test in determining the tensile rupture strain of Marshall specimens subjected to approximately the same testing conditions as employed in the current investigation. The early investigations of the ACHRP revealed that the percentage of asphalt in recovered asphalt cores of a particular section of highway was similar to the percentage of asphalt in the design mix for the same section of highway and hence the maximum measured strains were likely to approximate those reported by Gillespie and within a range where the high sensitivity of Tuckerman Strain Gauge would be desirable.

A gauge length of one inch was adopted throughout the testing program. The unknown location of the failure plane, which usually occurs in a band up to 1/2 inch on either side of the vertical axis, restricted the use of a smaller gauge length. The tensile stress at a

point 1/2 inch from the vertical axis can be calculated as being 78% of the maximum tensile stress and the strain measurements based on this inch, assuming a linear relationship in stress distribution, would result from a tensile stress that is approximately 90% of the maximum stress along the vertical axis. The error involved in the assumption that the measured strain is a result of maximum stresses increases as the gauge length becomes greater than 1 inch.

Fabricated, 1/4" x 1/4" x 1/8" aluminum blocks with scribed lines were fixed to the samples with asphalt cement. These blocks were placed one inch apart along the horizontal axes and on either side of the vertical axes of the samples. The knife edges of the Tuckerman Strain Gauge were placed in the scribed lines and the gauge was held in place with an elastic band attached to the gauge and encircling the sample.

The Tuckerman Strain Gauge is read by means of an autocollimator, the main components of which are; an eyepiece, a reticule, a fiduciary line and vernier, and a lamp for illuminating the fiduciary line and vernier. The total horizontal strain is calculated by the following equation; (presented in the Instruction Manual No. 75, 1958)

$$S = \frac{FALR}{Ex1000}$$

where:

S is strain in inches per inch

R equals the elapsed fiduciary reading from the beginning of the test in complete divisions

F, A, L and E are constants depending on the strain gauge used.

The destructive nature of the tensile splitting test necessitated the removal of the strain gauge prior to failure of the samples and therefore failure strains could not be directly determined by experimental procedure. An indirect method of determining approximate rupture strains of the samples was developed by relating the elapsed time from the beginning of a test to strain measurements taken prior to failure.

Complete details of the Tuckerman Strain Gauge and the indirect method of estimating rupture strain are contained in Appendix B and Appendix C respectively.

Temperature Control

The entire testing equipment, with the exception of the self-balancing load indicator, was placed in a temperature controlled room. The room was equipped with heavy insulated doors and a refrigerating unit capable of temperatures as low as -40°F. The room was kept at a temperature of 0°F (± 3°F) throughout the investigation, the temperature being continuously recorded by means of a Brown Continuous Balance Temperature Recorder.

The asphalt concrete samples were stored in the room for at least 24 hours before testing. The self-balancing load indicator was placed in an adjacent room, the two rooms being served by an intercom system.

Recording of Data

A two man operation was required for the recording of the data. Readings of vertical movement of the loading platform of the compression machine and fiduciary readings were taken in the temperature controlled room and relayed to the adjacent room by means of the intercom system. These readings were taken every five divisions transversed by the load indicator (approximately every 600 lb increment of load). The time elapsed from the beginning of the test to each of these readings was also recorded.

As previously mentioned, the Tuckerman Strain Gauge and the vertical strain dial were removed from their positions prior to failure of the samples and therefore only load and time were recorded at failure.

A detailed testing procedure, sample data sheets and example calculations are presented in Appendix C.

Examination of Samples After Testing

Each sample was inspected after failure to observe any characteristics which could be used to explain the test results. A number of such characteristics are: (1) the presence of large voids (2) the size and type of aggregate fracture and (3) the variation in density and thickness of the asphalt cores.

CHAPTER V

PRESENTATION OF TEST RESULTS

This Chapter contains the results of the testing program. The results of numerous tests performed on asphalt concrete cores extracted from a particular section of highway were averaged. Similarly, the tensile strength characteristics of Marshall specimens composed of asphalt from a single source were also averaged. These average results are presented in the following figures and tables. The results of individual tests have been tabulated and are presented in Appendix D. A brief description of each figure and table, included in this chapter, follows.

TABLE II shows the average tensile properties of the asphalt concrete cores extracted from the three sections of highway 21-A-1.

The terms contained in TABLE II and the remaining tables presented in this Chapter are defined as follows:

Average Cracks Per Mile - the frequency of transverse cracking occurring in a section of highway, as determined from field surveys, expressed as the number of transverse cracks in a one mile section of highway.

Maximum Stress - the maximum tensile stress developed within a sample as determined by the equation $S_x = 2P/\pi dt$. (In all tests the

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TABLE II

AVERAGE TEST RESULTS OF ASPHALT CORES*

HIGHWAY 21-A-1

	Section A (10-150/200)	Section B (10-150/200)	Section C (10-150/200)
Average Cracks Per Mile	229	72	0
Thickness of Core (ins)	1.72	2.12	1.62
Density of Core (pcf)	144.3	145.6	142.7
Time to Failure (mins-secs)	4 - 35	4 - 17	4 - 45
Maximum Stress (psi)	409	456	400
Proportional L. Stress (psi)	266	310	329
Elastic Strain (in/in x 10^{-5})	16	20	20
Plastic Strain (in/in x 10^{-5})	41	62	24
Strain at Failure (in/in x 10^{-5})	57	82	44
Rate of Loading (in/min)	.054	.054	.055
Stiffness Modulus (psi x 10^3)	953	649	1060

* Results presented are averages of five cores per section

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maximum tensile strength corresponded to the ultimate tensile strength.)

Proportional Limit Stress - that tensile stress at which the tensile strain ceases to be proportional to the tensile stress, as determined from tensile stress-strain plots.

Elastic Strain - the tensile strain at the proportional limit.

Plastic Strain - the difference between the tensile strain at failure and the elastic strain, as determined from tensile stress-strain plots.

Strain at Failure - the maximum tensile strain which a sample can withstand and as determined by an indirect method relating time elapsed from the beginning of a test to tensile-plastic strain prior to failure. (The method of determining tensile failure strain is presented in Appendix C.)

Rate of Loading - the rate of vertical deformation to which the samples are subjected.

Stiffness Modulus - the ratio of maximum tensile stress to the maximum tensile strain. (A secant modulus at failure conditions, as determined from tensile stress-strain plots.)

FIGURE 4 shows the average tensile stress-strain relationships of the asphalt concrete cores extracted from the three sections of highway 21-A-1.

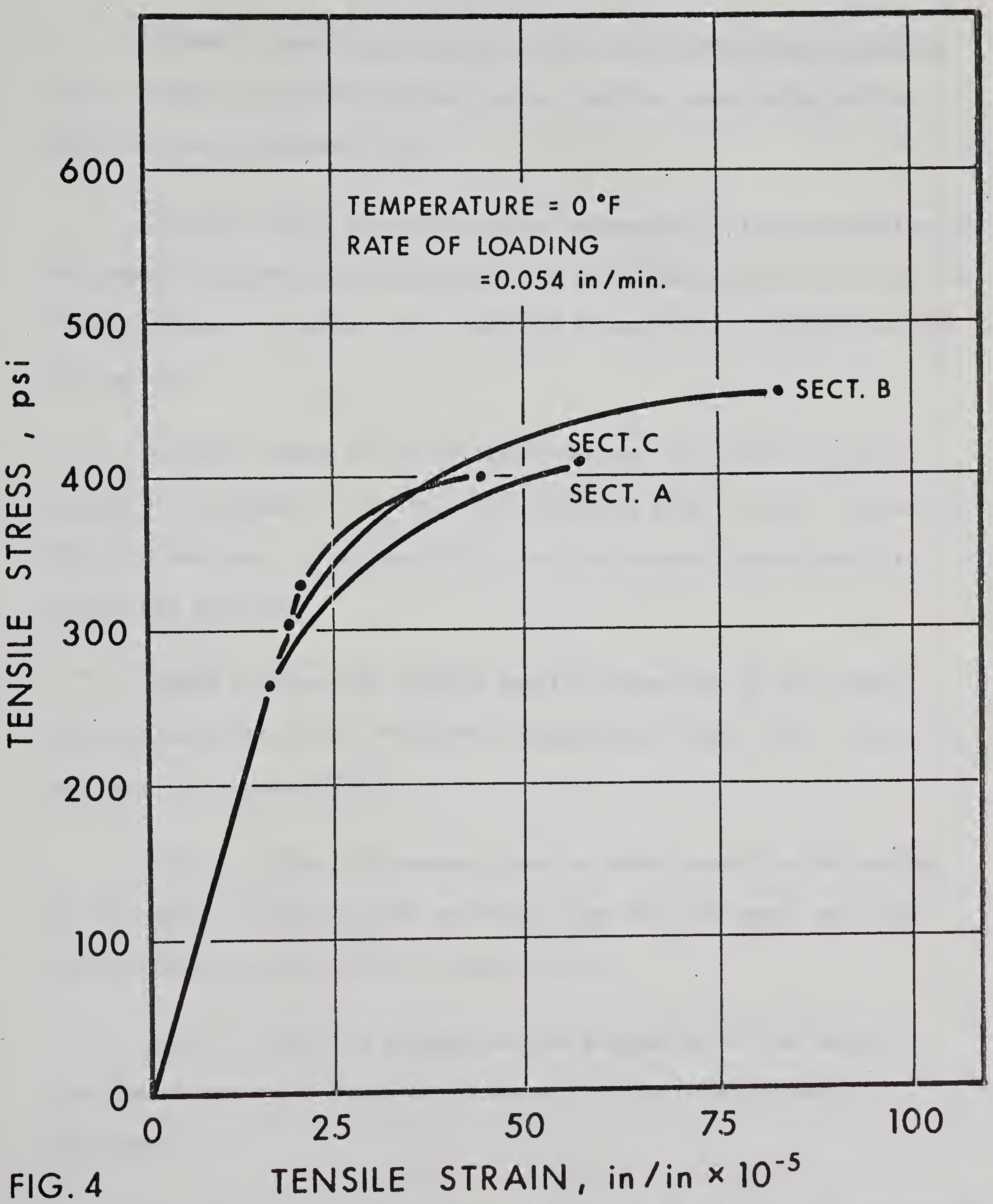

FIG. 4
AVERAGE TENSILE STRESS - STAIN IN RELATIONSHIP, HIGHWAY 21-A-1

TABLE III shows the average tensile properties of the asphalt concrete cores extracted from the five sections of highway 34-A-1.

FIGURE 5 shows the average tensile stress-strain relationships of the asphalt concrete cores extracted from the upper course of the five sections of highway 34-A-1.

FIGURE 6 shows the relationship between the stiffness modulus of the asphalt concrete cores from both the upper and lower courses, of the five sections, of highway 34-A-1 and the average cracks per mile within the sections.

FIGURE 7 shows the relationship between the tensile failure strain of the asphalt concrete cores extracted from the upper course, of the five sections, of highway 34-A-1 and the average cracks per mile within the sections.

TABLE IV shows the average tensile properties of the asphalt concrete cores extracted from both the upper and lower lifts, of the two sections, of highway 57-A.

FIGURE 8 shows the average tensile stress-strain relationships of the asphalt concrete cores extracted from both the upper and lower lifts, of the two sections, of highway 57-A.

TABLE V shows the average tensile properties of the Marshall specimens composed of asphalts obtained from the three different suppliers.

TABLE III

AVERAGE TEST RESULTS OF ASPHALT CORES*

HIGHWAY 34-A-1

	Lower Course (5-150/200)					Upper Course (1-200/300)				
	Section					Section				
	G-1	G-2	G-3	G-4	G-5	G-1	G-2	G-3	G-4	G-5
Average Cracks Per Mile	350	540	235	45	100	350	540	235	45	100
Thickness of Core (ins)	2.09	1.73	2.28	2.09	1.85	1.99	1.91	2.08	2.03	1.89
Density of Core (pcf)	145.3	144.0	145.0	141.6	143.8	146.7	144.8	142.9	144.8	143.4
Time to Failure (mins-secs)	4-58	5-01	5-06	4-49	5-54	5-22	5-15	5-10	4-32	5-08
Maximum Stress (psi)	451	430	514	380	434	578	540	498	412	393
Proportional L. Stress (psi)	211	186	298	41	217	387	426	318	272	257
Elastic Strain (in/inx10^{-5})	17	17	28	6	21	17	24	17	22	21
Plastic Strain (in/inx10^{-5})	115	68	52	129	81	47	16	70	66	75
Strain at Failure (in/inx10^{-5})	132	135	79	135	102	64	40	87	88	96
Rate of Loading (in/min)	.055	.056	.057	.058	.055	.054	.056	.056	.057	.053
Stiffness Modulus (psi x 10^3)	342	520	653	292	419	930	1350	550	464	412

* Results presented are averages of two cores per section

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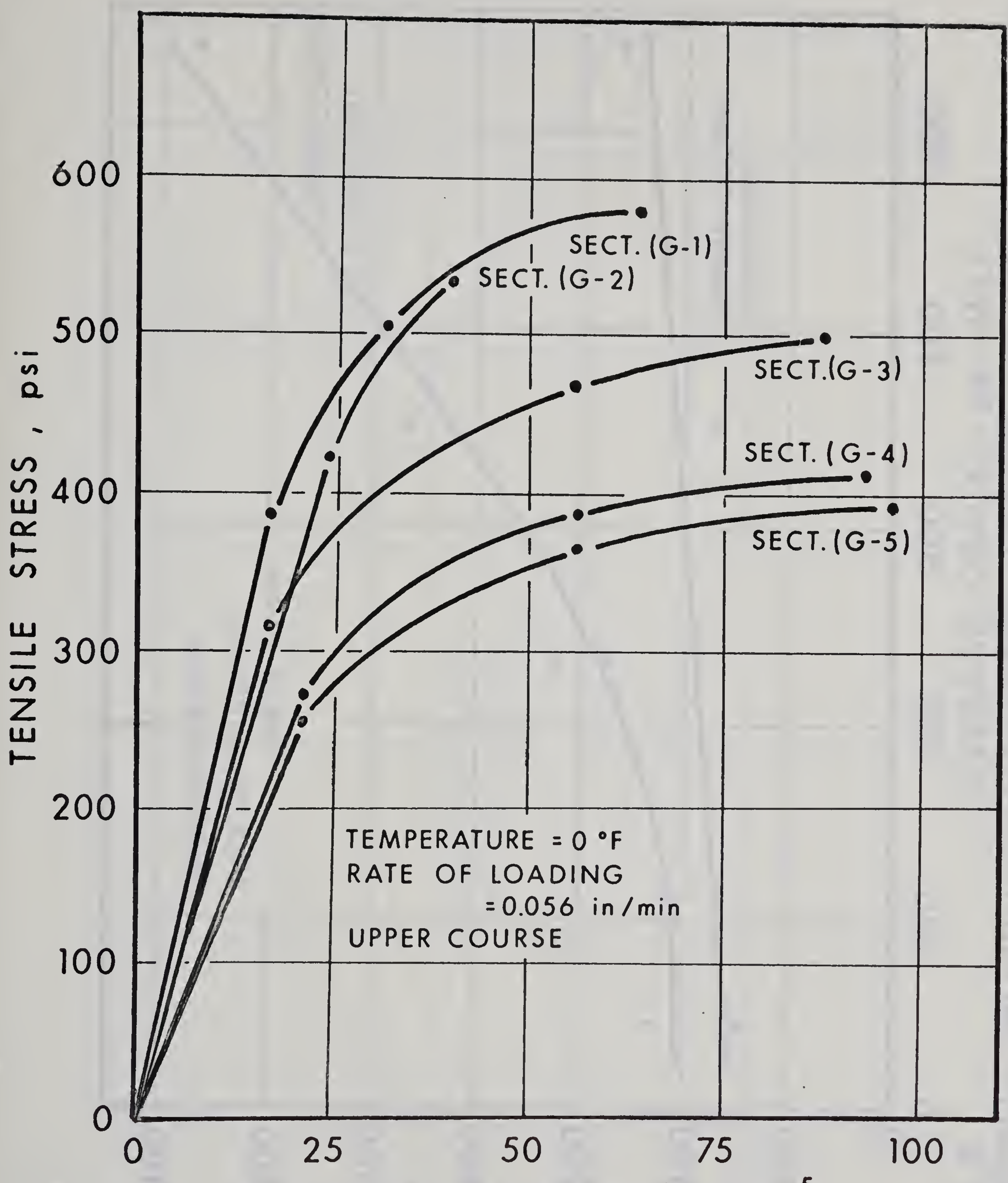


FIG. 5

AVERAGE TENSILE STRESS-STRAIN RELATIONSHIPS
HIGHWAY 34-A-1

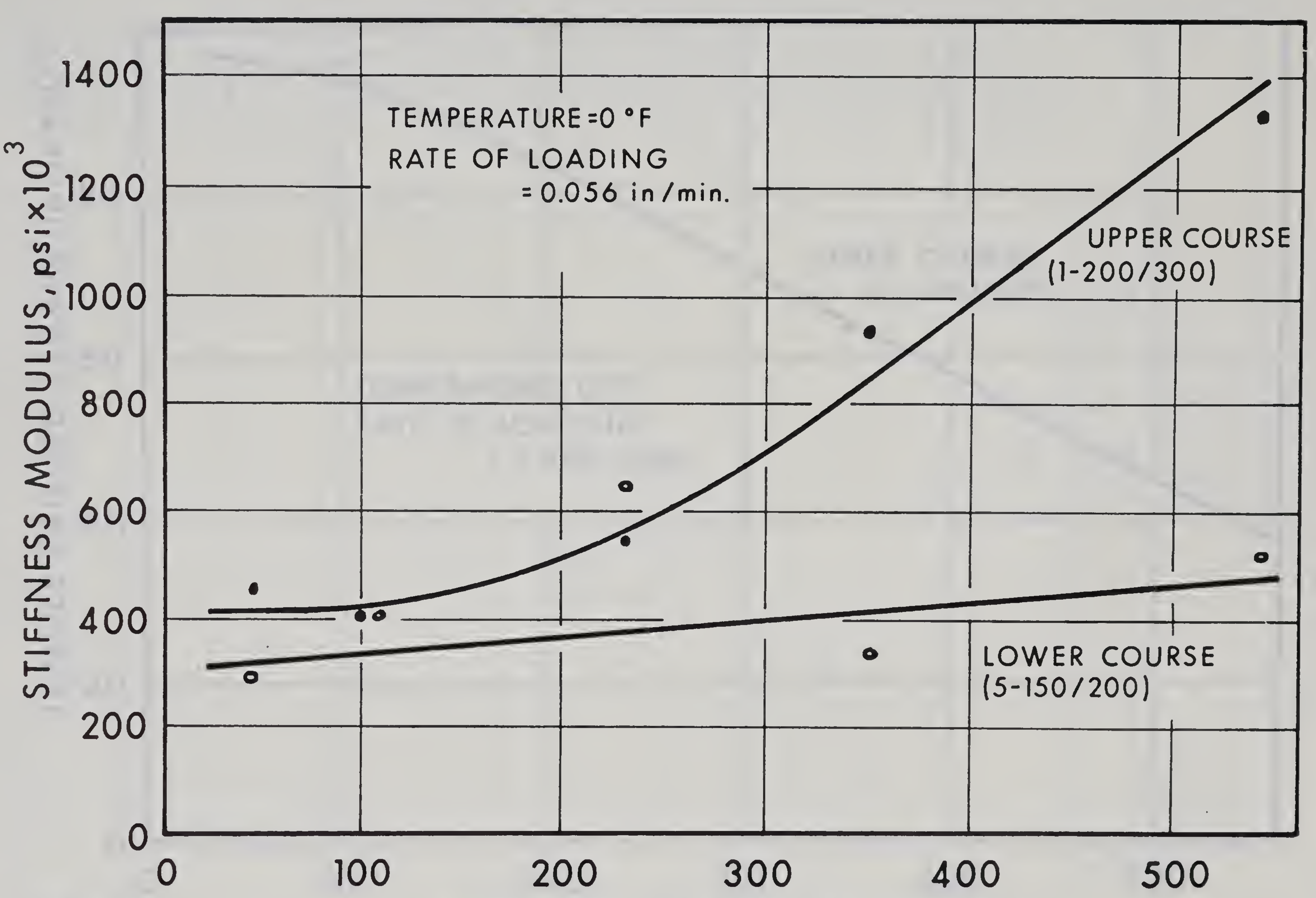
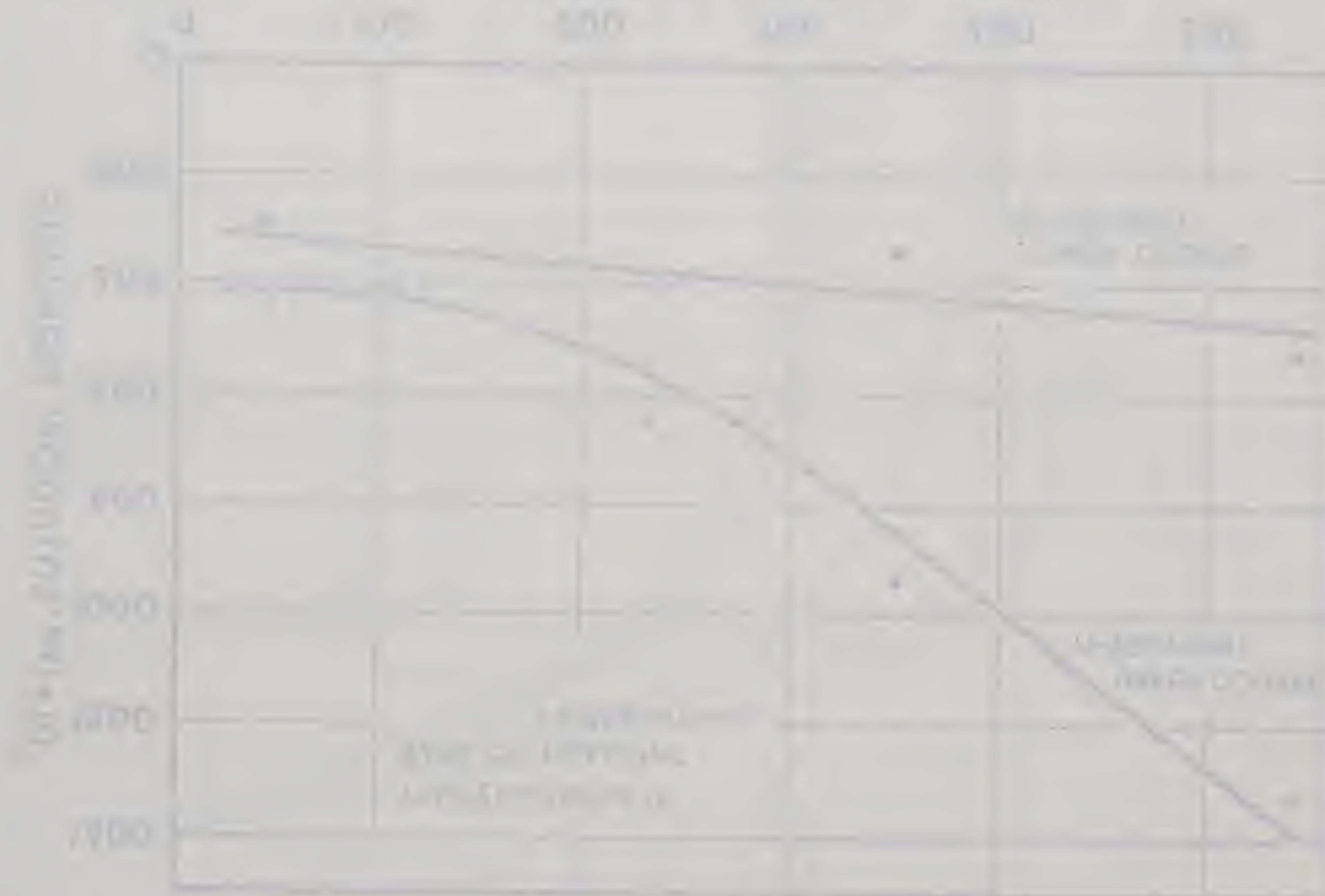


FIG. 6
STIFFNESS MODULUS vs AVERAGE CRACKS per MILE, HIGHWAY 34-A-1



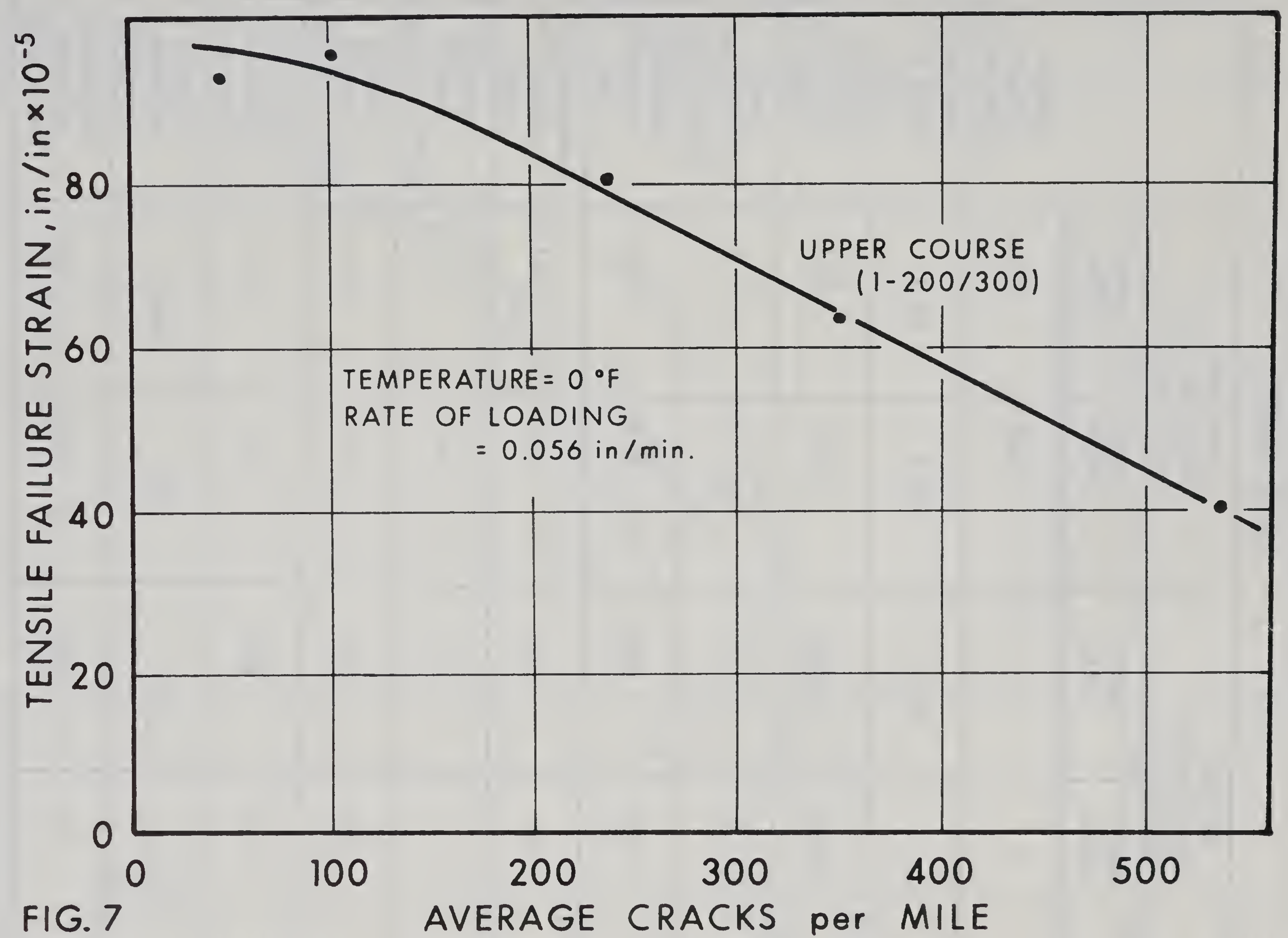


FIG. 7
TENSILE FAILURE STRAIN vs AVERAGE CRACKS per MILE, HIGHWAY 34-A-1

TABLE IV

AVERAGE TEST RESULTS OF ASPHALT CORES

HIGHWAY 57-A

	Section (12.54-14.54) (1-150/200)		Section (14.54-16.55) (5-150/200)	
	Lower Lift (1)	Upper Lift (1)	Lower Lift (2)	Upper Lift (2)
Average Cracks Per Mile	348	348	0	0
Thickness of Core (ins)	1.69	2.35	2.14	2.18
Density of Core (pcf)	138.2	142.8	143.0	142.2
Time to Failure (mins-secs)	4 - 23	5 - 02	4 - 57	4 - 49
Maximum Stress (psi)	296	413	426	395
Proportional L. Stress (psi)	156	259	144	49
Elastic Strain (in/in x 10^{-5})	12	18	13	7
Plastic Strain (in/in x 10^{-5})	63	57	87	104
Strain at Failure (in/in x 10^{-5})	75	75	100	111
Rate of Loading (in/min)	.054	.055	.052	.054
Stiffness Modulus (psi x 10^3)	445	557	428	376

(1) Results presented are averages of four cores per Lift.

(2) Results presented are averages of five cores per Lift.

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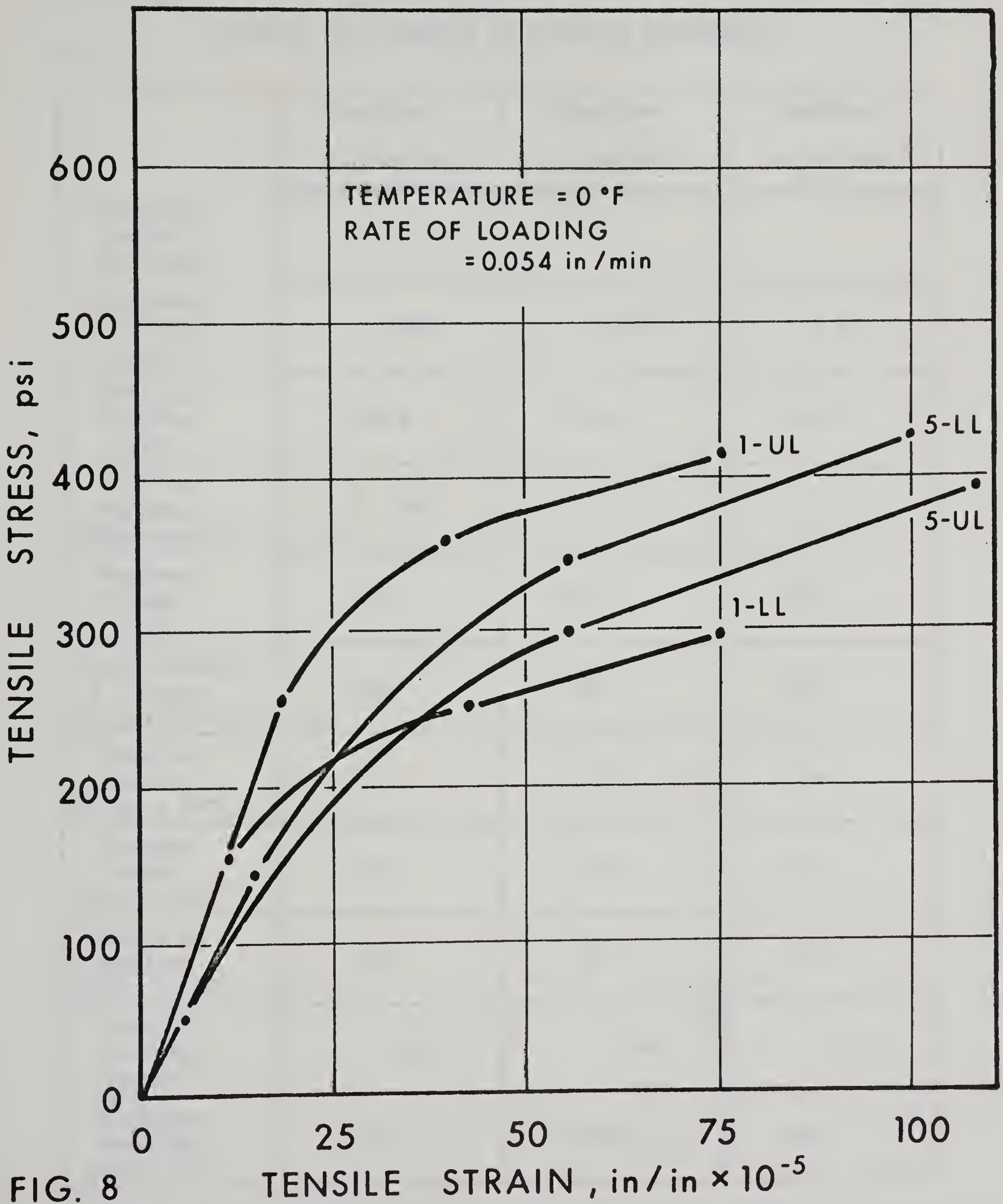

FIG. 8

AVERAGE TENSILE STRESS-STRAIN RELATIONSHIPS HIGHWAY 57-A

TABLE V

AVERAGE TEST RESULTS OF MARSHALL SPECIMENS*

	Supplier (1-200/300)	Supplier (2-200/300)	Supplier (3-200/300)
Average Cracks Per Mile	-	-	-
Thickness of Core (ins)	2.50	2.50	2.50
Density of Core (pcf)	144.5	144.6	145.3
Time to Failure (mins-secs)	5 - 38	5 - 39	5 - 31
Maximum Stress (psi)	560	538	551
Proportional L. Stress (psi)	396	344	352
Elastic Strain (in/in x 10^{-5})	22	28	30
Plastic Strain (in/in x 10^{-5})	18	39	31
Strain at Failure (in/in x 10^{-5})	40	67	61
Rate of Loading (in/min)	.052	.054	.055
Stiffness Modulus (psi x 10^3)	1437	893	921

* Results presented are averages of five specimens per supplier

FIGURE 9 shows the average tensile stress-strain relationships of the Marshall specimens composed of asphalts from the three different suppliers.

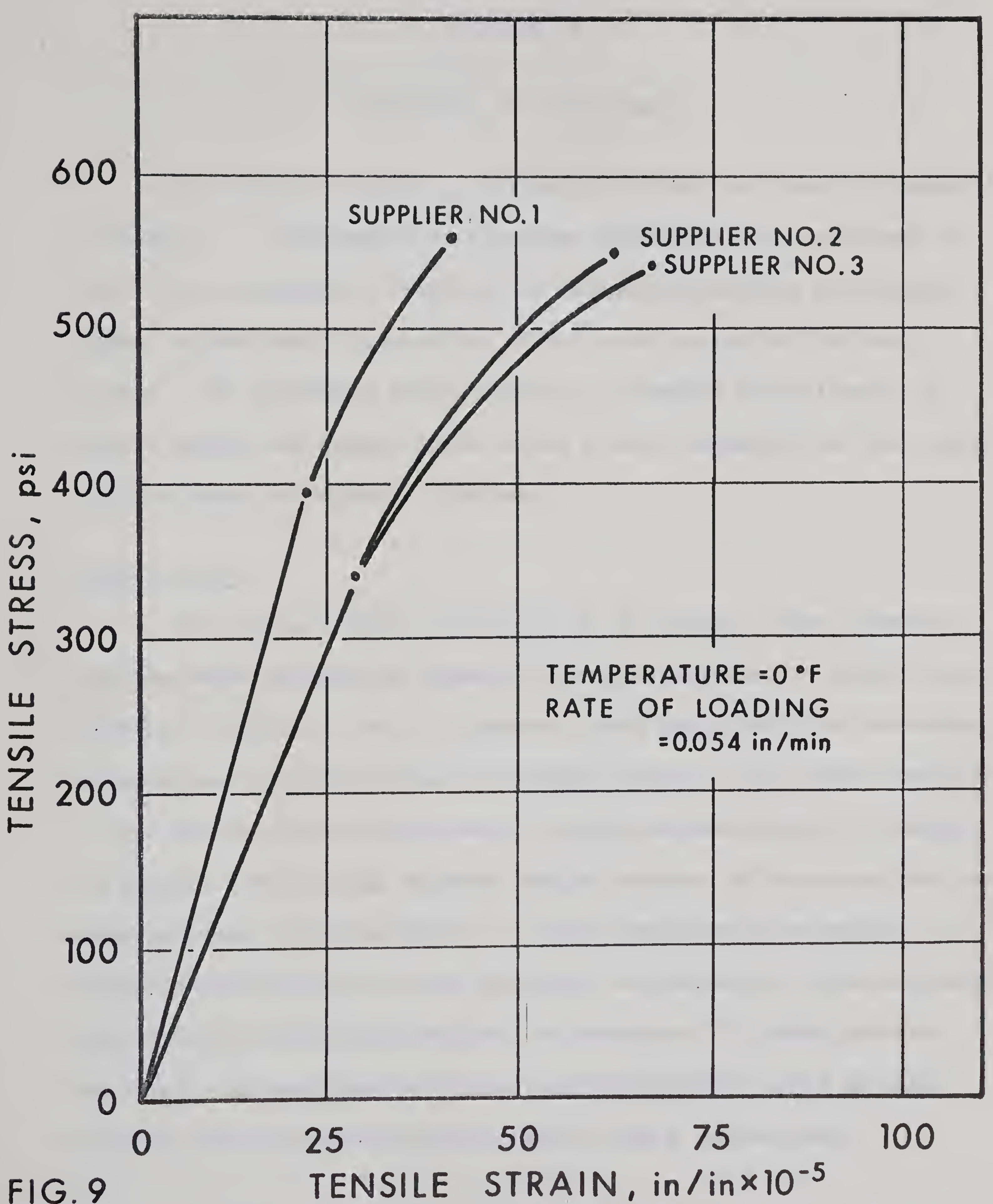

FIG. 9
AVERAGE TENSILE STRESS-STRAIN RELATIONSHIPS
MARSHALL SPECIMENS

CHAPTER VI

DISCUSSION OF TEST RESULTS

This Chapter contains a discussion of the test results presented in Chapter V. Each highway is discussed individually in an attempt to relate the variations in frequency of transverse cracking within each highway to the tensile properties of the cores extracted from each section. The discussion also attempts to interpret the influence of asphalt source and asphalt grade on the tensile properties of the asphalt concrete cores and Marshall specimens.

Highway 21-A-1

The average tensile properties of the asphalt cores extracted from the three sections of highway 21-A-1, as presented in TABLE II and FIGURE 4 of Chapter V, show no apparent relationship with the occurrence of transverse cracking within the various sections. The elastic portion of the tensile stress-strain curves for the various sections of highway are similar. The average maximum tensile stresses of the cores from the three sections vary from 400 psi for cores extracted from section C, which had exhibited zero cracks per mile, to 456 psi for cores extracted from section B, which had exhibited an average of 72 cracks per mile. The average maximum tensile strains vary from 44×10^{-5} in/in to 82×10^{-5} in/in for cores extracted from sections C and B, respectively.

The significance of the above results is questionable because of the large deviation in the tensile properties of the individual cores extracted from any one section of the highway. The range in values obtained from the testing program for each section of highway are as follows,

Section A:

Maximum Tensile Stress 260 to 515 (psi)

Maximum Tensile Strain 15 to 108 (10^{-5} in/in)

Section B:

Maximum Tensile Stress 410 to 502 (psi)

Maximum Tensile Strain 36 to 114 (10^{-5} in/in)

Section C:

Maximum Tensile Stress 344 to 454 (psi

Maximum Tensile Strain 25 to 67 (10^{-5} in/in)

The largest variation in tensile properties occurs in the values of maximum tensile strain. A possible factor contributing to the wide range in strain values, and hence the nonexistence of a relationship between the frequency of transverse cracking and the tensile properties of the asphalt cores, is the asphalt crude source. All the asphalt used in the construction of the three sections of highway was obtained from supplier No. 10. This supplier obtains crude from many different crude fields, the result being a possible wide variation in the properties of the manufactured asphalt which in turn are reflected in the tensile properties of the asphalt concrete cores.

Highway 34-A-1

The tensile properties of asphalt concrete cores extracted from both the upper and lower courses of five sections of highway 34-A-1 were determined. Each section exhibited a different transverse cracking frequency. The lower course was constructed using 150/200 penetration grade asphalt obtained from supplier No. 5 and the upper course was constructed using 200/300 penetration grade asphalt obtained from supplier No. 1. Prior to placement of the upper surface course the frequency of transverse cracking on the lower surface course had been negligible; however after placement of the present surface course the frequency of transverse cracking increased.

Comparing the average tensile properties of the lower course cores with the average tensile properties of the cores extracted from the upper course, as presented in TABLE III of Chapter V, it is seen that (1) the maximum tensile strengths of the cores extracted from the lower course approximate the maximum tensile strengths of cores extracted from the upper course and (2) the maximum tensile strains of cores extracted from the lower course are as much as 200 percent greater than the maximum tensile strains of the upper course cores. These two properties combined result in lower failure stiffness modulii for cores extracted from the lower course as compared to cores extracted from the upper course.

The greater maximum tensile strain and the resulting lower stiffness of the lower course suggests that the occurrence of transverse

cracking within the sections of highway is mainly related to the tensile properties of the upper course. The differences in the maximum tensile strains of the two courses appear to be associated with the asphalt supplier. Asphalt concrete mixes composed of the higher penetration grade asphalt supposedly should exhibit greater tensile strains at low temperatures as compared to mixtures composed of the lower penetration grade asphalt. The test results indicate a behaviour opposite to this, and hence the variation in maximum tensile strain of the two courses could be directly associated with the asphalt supplier by not taking into account any variations which may be due to differences in the physical properties of the cores tested.

The test results suggest that the occurrence of transverse cracking is related to the tensile properties of the upper course. The following paragraphs discuss the tensile properties of the cores extracted from the five sections of the upper course in relation to the occurrence of transverse cracking within the sections.

Comparing the average tensile stress-strain properties of the cores extracted from the five sections of the upper course, as presented in FIGURE 5 of Chapter V, to the occurrence of transverse cracking within the sections the following general trends are observed. (1) The modulus of elasticity increases as the frequency of transverse cracking increases, (2) the maximum tensile stress increases as the frequency of transverse cracking increases and (3) the maximum tensile strain decreases as the frequency of transverse cracking increases.

The modulus of elasticity, in this thesis, refers to the ratio of the tensile stress at the proportional limit to the elastic tensile strain at the proportional limit. The elastic tensile strain for all sections is approximately 20×10^{-5} in/in and therefore the increase in modulus of elasticity corresponds to an increase in the tensile stress at the proportional limit. The average tensile stress at the proportional limit varies from 257 psi to 426 psi. These values represent section G-5 which exhibited an average of 100 cracks per mile and section G-2 which exhibited an average of 540 cracks per mile, respectively.

The average maximum tensile stress varies from 393 psi, representing section G-5, to 578 psi, representing section G-1. This variation in maximum tensile stress is only slightly greater than the variation occurring at the proportional limit.

The average tensile failure strains, representing each section of the upper course, show the largest variation of all tensile properties. The average tensile failure strain varies from 40×10^{-5} in/in, representing section G-2, to 96×10^{-5} in/in, representing section G-5.

Combining the maximum tensile stress with the maximum tensile strain, representing each section of highway, results in a relationship between the stiffness modulus for each section and the average number of cracks per mile within the sections. This relationship is shown in FIGURE 6 of Chapter V. As the stiffness modulus increases the average number of cracks per mile also increases. This figure also shows that the tensile properties of the upper course have a greater influence on

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the occurrence of transverse cracking than do the tensile properties of the lower course. As the number of average cracks per mile decrease the stiffness modulus of the cores extracted from the upper course tends to approach the stiffness modulus of cores extracted from the lower course. For zero cracks per mile the stiffness modulus of the upper course would be approximately 400,000 psi. This value corresponds closely to 450,000 psi; the average stiffness modulus of the lower course.

As previously mentioned, the tensile failure strain of the cores representing the upper course show the greatest variation of all tensile properties. FIGURE 7 shows that there is a curvilinear relationship between the average tensile failure strains of cores extracted from the upper course and the average cracks per mile in the five sections of highway investigated. The extension of the straight line portion of this relationship to the point representing zero cracks per mile indicates a tensile failure strain of approximately 110×10^{-5} in/in. This value corresponds closely to 120×10^{-5} in/in; the average tensile failure strain of the lower course.

In summary, the occurrence of transverse cracking on highway 34-A-1 appears to be associated with the tensile properties of the upper course. The most variable of these tensile properties is the tensile failure strain which tends to be influenced by the asphalt source.

<u>Highway 57-A</u>

The tensile properties of asphalt concrete cores extracted from both the upper and lower lifts of two sections of highway 57-A were

determined. The test results of each section are discussed individually and then in conjunction with one another. The results are also compared to those obtained on highway 34-A-1. The data referred to in the following paragraphs is presented in TABLE IV and FIGURE 8 of Chapter V.

The tensile stress-strain properties of the cores extracted from the upper and lower lifts of the section constructed of asphalt obtained from supplier No. 1 are similar with the exception of the maximum tensile stresses. The maximum tensile stress representing the upper lift is 413 psi while the maximum tensile stress of the lower lift is 296 psi. This variation in maximum stress is believed to be due to a stripping phenomena. Inspection of the lower lift cores after testing revealed that the majority of fracture took place at the asphalt-aggregate interface and the majority of aggregate was unfractured. All other cores, representing sections of the various highways, exhibited aggregate fracture. A possible cause of the stripping is water being entrapped between the asphalt bound base, on which the lower lift was constructed, and the upper lift. The water would tend to decrease the adhesion of the asphalt to the aggregate particles; the result being a decrease in maximum tensile stress. Similar occurrences of internal stripping have been reported by Fromm, Phang and Noga (1966).

The tensile stress-strain properties of the cores extracted from the upper and lower lifts of the section constructed of asphalt obtained from supplier No. 5 are similar. The small differences in the tensile properties which do exist are probably of no significance because of the limited number of cores tested per lift.

In comparing the two sections, the cores extracted from the section constructed of asphalt obtained from supplier No. 5 exhibit greater maximum tensile strains and also a more predominate plastic behaviour than the cores representing the section constructed of asphalt obtained from supplier No. 1. These two characteristics, together with the stripping phenomena, are probably the cause of the wide variation in the occurrence of transverse cracking within the two sections. Both sections were constructed using 150/200 penetration grade asphalt and therefore the differences in maximum tensile strain and plastic behaviour of the two sections are probably a direct reflection of the influence of asphalt source on the tensile properties of the pavement composite.

The average stiffness modulus of the cores extracted from the section exhibiting zero cracks per mile is 402,000 psi. This value corresponds favourably with 450,000 psi, the average stiffness modulus of lower course of highway 34-A-1. The section of highway 57-A and the lower course of highway 34-A-1 were both constructed using 150/200 grade asphalt obtained from supplier No. 5. Although stiffness modulii appear to be related to asphalt source, other factors such as variations in aggregate source, aggregate gradation and the physical properties of the two sections may influence this relationship.

The average stiffness modulus and average tensile failure strain of the cores extracted from the section of highway composed of asphalt obtained from supplier No. 1 are 557,000 psi* and 75×10^{-5} in/in,

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* Average stiffness modulus of lower lift not included in this average due to stripping phenomena.

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respectively. This section of highway had exhibited an average of 348 cracks per mile. Entering these values into FIGURE 6 and FIGURE 7 respectively, indicates that the use of a higher penetration grade asphalt results in lower stiffness modulii and greater maximum tensile strains than asphalt pavements composed of a lower penetration grade asphalt, obtained from the same supplier. This result suggests that the occurrence of transverse cracking may be reduced by use of a higher penetration asphalt, as compared to a low penetration grade asphalt, obtained from the same asphalt supplier.

The Marshall Specimens

The purpose of testing the Marshall specimens was to determine the influence of asphalt source on the low temperature tensile properties of asphalt concrete mixtures. The specimens were composed of 200/300 penetration grade asphalt obtained from three different suppliers. All specimens had the same physical properties and an asphalt content of 5.5 percent, by dry weight of aggregate. The results referred to in the following paragraphs are presented in TABLE V and FIGURE 9 of Chapter V.

The Marshall specimens composed of asphalt obtained from supplier No. 1 show greater tensile stresses at the proportional limit, lower elastic tensile strains and lower tensile failure strains, as compared to the specimens representing suppliers Nos. 2 and 3. The tensile properties of the Marshall specimens composed of asphalt from suppliers Nos. 2 and 3 are nearly identical.

These results suggest that the low temperature properties of asphalt concrete mixes are dependent on the asphalt source and the maximum tensile strain appears to be the property most affected by changes in the asphalt source.

Significance of Test Results

The significance of the test results is limited by the number of samples tested, the unknown physical properties of the cores such as asphalt content and percent of voids, the representativeness of the cores, the indirect method used in determining the tensile failure strain and the assumptions associated with the theory of the tensile splitting test. Because of these factors it is possible to relate the tensile properties of individual cores to any section of highway. However by categorizing the cores into their respective sections definite differences in the tensile properties of the sections are made evident.

Results of Other Investigators

The following paragraphs briefly review the findings of other investigations in relation to the occurrence of transverse cracking and the results presented in Chapter V.

Various investigations have determined maximum tensile strengths for asphalt concrete at temperatures in the order of 20°F. The decrease in tensile strength at lower temperatures has been postulated as being due to stress concentrations. Transverse cracking is generally believed to be associated with temperatures below 20°F and hence the possibility of thermally induced stresses exceeding the low temperature tensile strength of asphalt concrete mixtures is increased.

The theoretical solution of the tensile splitting test is based on the theory of elasticity. Investigations have shown that an asphalt concrete mix can be considered a predominately elastic material at temperatures below approximately 30°F. At low temperatures the rate of loading has been found to have little effect on the tensile properties of asphalt concrete mixes. The effect of small variations in the rate of loading on the tensile stress-strain relationships presented in Chapter V is therefore considered negligible.

The test results of this investigation suggest that the low temperature tensile properties of asphalt concrete cores are influenced by asphalt source and asphalt grade. Gillespie (1966), obtained similar results on Marshall specimens. Results of field investigations, reported by Culley (1966), also indicate that asphalt source and asphalt grade are significant factors related to transverse cracking of asphaltic concrete pavements.

The low temperature tensile properties of the Marshall specimens differed from those reported by Gillespie (1966). The maximum tensile stresses are approximately 40 percent greater and the maximum tensile strains are approximately 200 percent less than those found by Gillespie. Variation of temperature control in the two testing procedures is believed to be the main factor causing these differences. In the current investigation all specimens were tested at 0°F, in a temperature controlled room. Gillespie cooled the specimens to the test temperature and then tested the specimens at room temperatures. The increase in temperature of the specimens, during testing at room temperatures,

would cause lower maximum tensile stresses and greater maximum tensile strains, as compared to those obtained by the temperature controlled testing procedure.

An attempt was made to correlate the average stiffness modulus for the various sections of highways to the stiffness of a mix as determined by the nomograph developed by Heukelom and Klomp (1964). The stiffness modulii determined from the nomograph were generally greater than those measured and did not vary consistently with the measured values. A possible reason for this discrepancy is that the nomograph was developed assuming a low void content, well compacted design mix, whereas the percentage of voids in the pavement surfaces investigated were greater than those in original mix design, as reported by Shields and Anderson (1964).

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

The purpose of this study was to use the tensile splitting test to determine the low temperature tensile properties of asphalt concrete cores extracted from various highways and to relate these properties to the occurrence of transverse cracking within the highways. The influence of asphalt source and asphalt grade on the tensile properties of the asphalt cores and Marshall specimens was also investigated. The conclusions and recommendations arising from these investigations are presented in this Chapter.

Conclusions

1. The tensile splitting test is an applicable test method for determining the low temperature tensile properties of asphaltic mixtures. The advantages of the test are, (1) the relatively simple expression defining the stress conditions, (2) the rapid rate of conducting the test and (3) the simplicity of shape of the samples.

2. The occurrence of transverse cracking is dependent upon the low temperature tensile properties of the asphalt concrete pavement and varies as follows:

(a) The occurrence of transverse cracking increases as the tensile failure strain decreases.

(b) The occurrence of transverse cracking increases as the stiffness modulus increases.

(c) The occurrence of transverse cracking increases as the modulus of elasticity increases.

3. The low temperature tensile properties of asphalt concrete pavements are dependent upon the asphalt source and the asphalt penetration grade.

(a) Variations in asphalt source and asphalt grade give rise to variations in maximum tensile strains and stiffness modulii.

(b) Asphalt pavements and Marshall specimens composed of asphalts from different sources but of the same penetration grade exhibit different low temperature tensile properties.

(c) Asphalt pavements composed of a high penetration asphalt do not necessarily exhibit greater failure strains and lower stiffness modulii than asphalt pavements composed of a low penetration grade asphalt. The effect of variations in asphalt penetration grade on the low temperature tensile properties of asphalt pavements appears to be dependent on the asphalt source.

Recommendations

1. The testing procedure for performing the tensile splitting test should be improved by using apparatus capable of continuous strain recording.

2. The low temperature tensile properties of future mix designs should be investigated. These properties should then be related to the low temperature tensile properties of the corresponding asphalt pavement surface and to the frequency of transverse cracking within the surface. In this manner a design method may be established whereby the occurrence of transverse cracking could be reduced.

3. Field investigations for determining the temperature at which cracking occurs are required. With this information a laboratory testing program, closely resembling field conditions, could be established.

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LIST OF REFERENCES

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APPENDIX A

THE TENSILE SPLITTING TEST

THE TENSILE SPLITTING TEST

The theoretical solution of the tensile splitting test is based on the theory of elasticity. The stress distribution, assuming a plane stress condition, within a cylindrical disc subjected to concentrated loads along two opposite generatrices of the disc has been presented by Frocht (1949) and reviewed by Gillespie (1966).

FIGURE A1 shows a cylinder subjected to point loading and a co-ordinate system defining the location of an element, "A", within the cylinder.

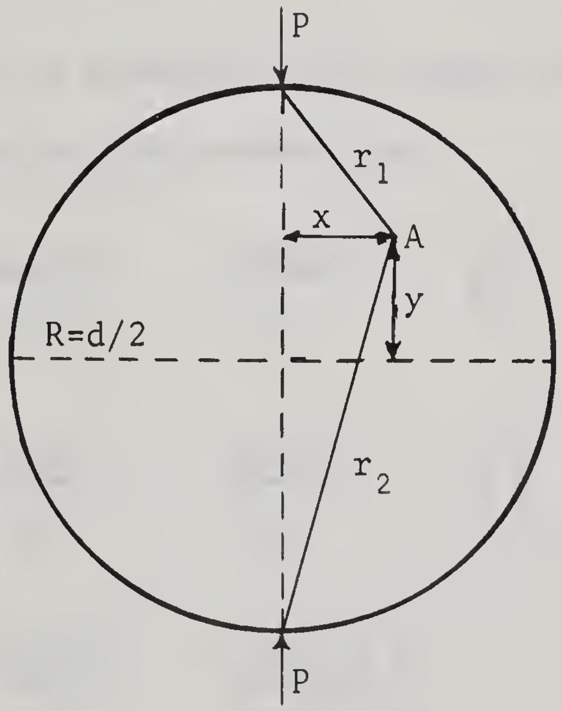

FIGURE A1. COORDINATE SYSTEM

Removing the element A from the cylinder and examining the component stresses that the element is subjected to results in FIGURES A2 and A3.

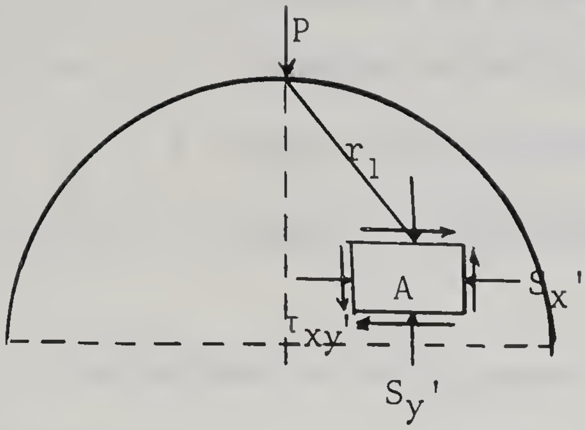

FIGURE A2

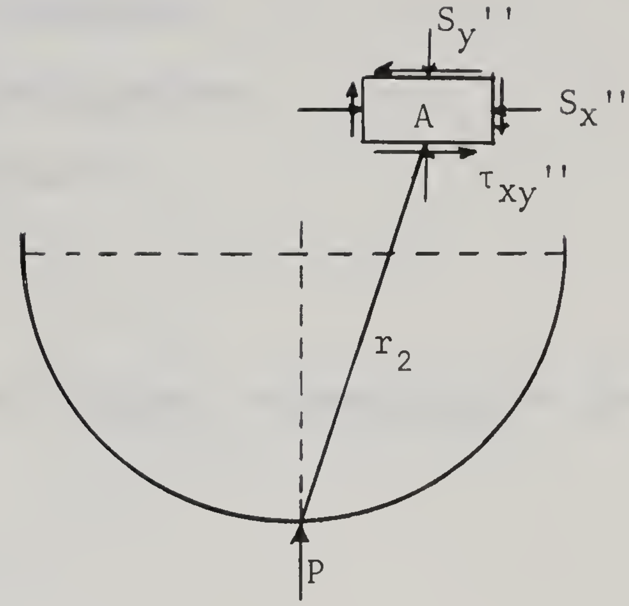

FIGURE A3

COMPONENT STRESSES

The general stress equations that express the tensile, compressive and shear stress on the element are:

$$S_x = \frac{-2P}{\pi t}\left[\frac{(R-y)x^2}{r_1{}^4} + \frac{(R+y)x^2}{r_2{}^4} - \frac{1}{d}\right] \quad \text{..........} \quad (1)$$

$$S_y = \frac{-2P}{\pi t}\left[\frac{(R-y)^3}{r_1{}^4} + \frac{(R+y)^3}{r_2{}^4} - \frac{1}{d}\right] \quad \text{..........} \quad (2)$$

$$\tau_{xy} = \frac{2P}{\pi t}\left[\frac{(R-y)^2x}{r_1{}^4} - \frac{(R+y)^2x}{r_2{}^4}\right] \quad \text{..........} \quad (3)$$

where:

S_x, S_y, τ_{xy} = stress components with respect to rectangular coordinates

x, y = rectangular coordinates

P = load applied to specimen

t = thickness of cylindrical specimen

d = diameter of the cylindrical specimen

R = radius of cylindrical specimen

r_1, r_2 = location coordinates

For points on the diameter of the cylinder, perpendicular to the applied load,

$$y = 0 \quad \text{and} \quad r_1 = r_2 = \sqrt{x^2+R^2}$$

the general stress equations simplify to

$$S_x = \frac{2P}{\pi dt}\left[\frac{d^2-4x^2}{d^2+4x^2}\right]^2 \qquad \dots\dots\dots\dots (4)$$

$$S_y = \frac{-2P}{\pi dt}\left[\frac{4d^4}{(d^2+4x^2)^2} - 1\right] \qquad \dots\dots\dots\dots (5)$$

$$\tau_{xy} = 0 \qquad \dots\dots\dots\dots (6)$$

The vertical stress, S_y, along the X axis is always a compressive stress and varies from $-6P/\pi dt$ at the centre to zero at the circumference. The accompanying horizontal stress, S_x, is a tensile stress and varies from $2P/\pi dt$ at the centre to zero at the circumference. This indicates that the material tested must have a compressive strength at least three times its tensile strength in order to ensure a tensile failure. The stress distribution along the X axis is shown in FIGURE A4.

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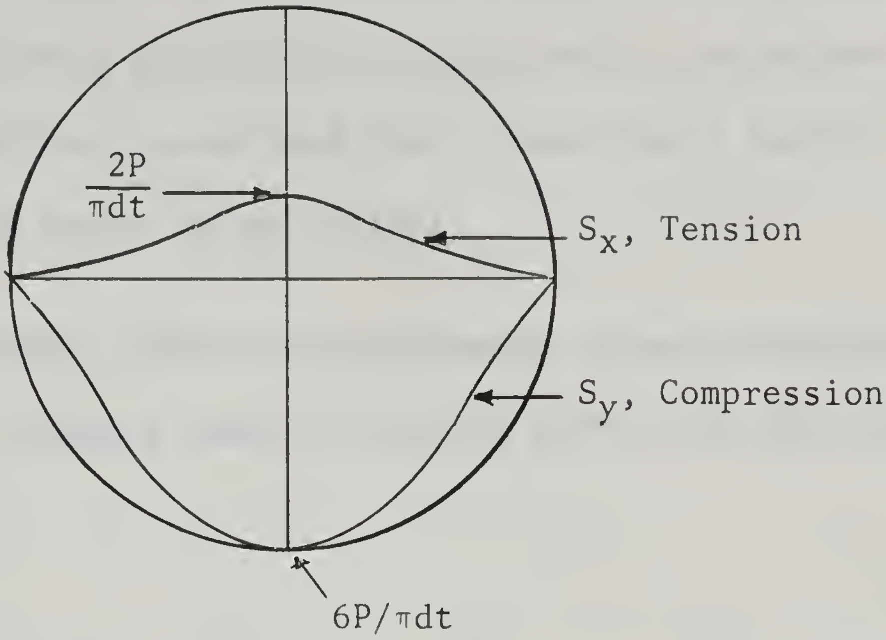

FIGURE A4. STRESS DISTRIBUTION ON X-AXIS

For points on a vertical plane, through the centre of the disc, the general stress equations simplify to:

$$S_x = \frac{2P}{\pi dt}$$

$$S_y = \frac{-2P}{\pi dt}\left[\frac{2}{d-2y} + \frac{2}{d+2y} - \frac{1}{d}\right]$$

$$\tau_{xy} = 0$$

The horizontal tensile stresses, S_x, along the vertical plane have a constant value of $2P/\pi dt$ and the vertical compressive stress, S_y, varies from $-6P/\pi dt$ at the centre to infinity at the load points. With a concentrated load failure will occur at the load points due to the high compressive stresses. It has been shown by photoelastic studies, Mitchell (1961), that this point of maximum stress can be moved away from the circumference by using a distributed load applied through a loading strip. The distributed load changes the S_x stress from tension

to compression in the vicinity of the loading plates and experimental evidence indicates that this condition is sufficient to retard the compressive failure at the load points and the cylinder fails due to tensile stresses at the centre Wright (1955).

According to Wright (1955), the horizontal stress distribution on the vertical plane, using a loading strip of width less than d/10, is shown in FIGURE A5.

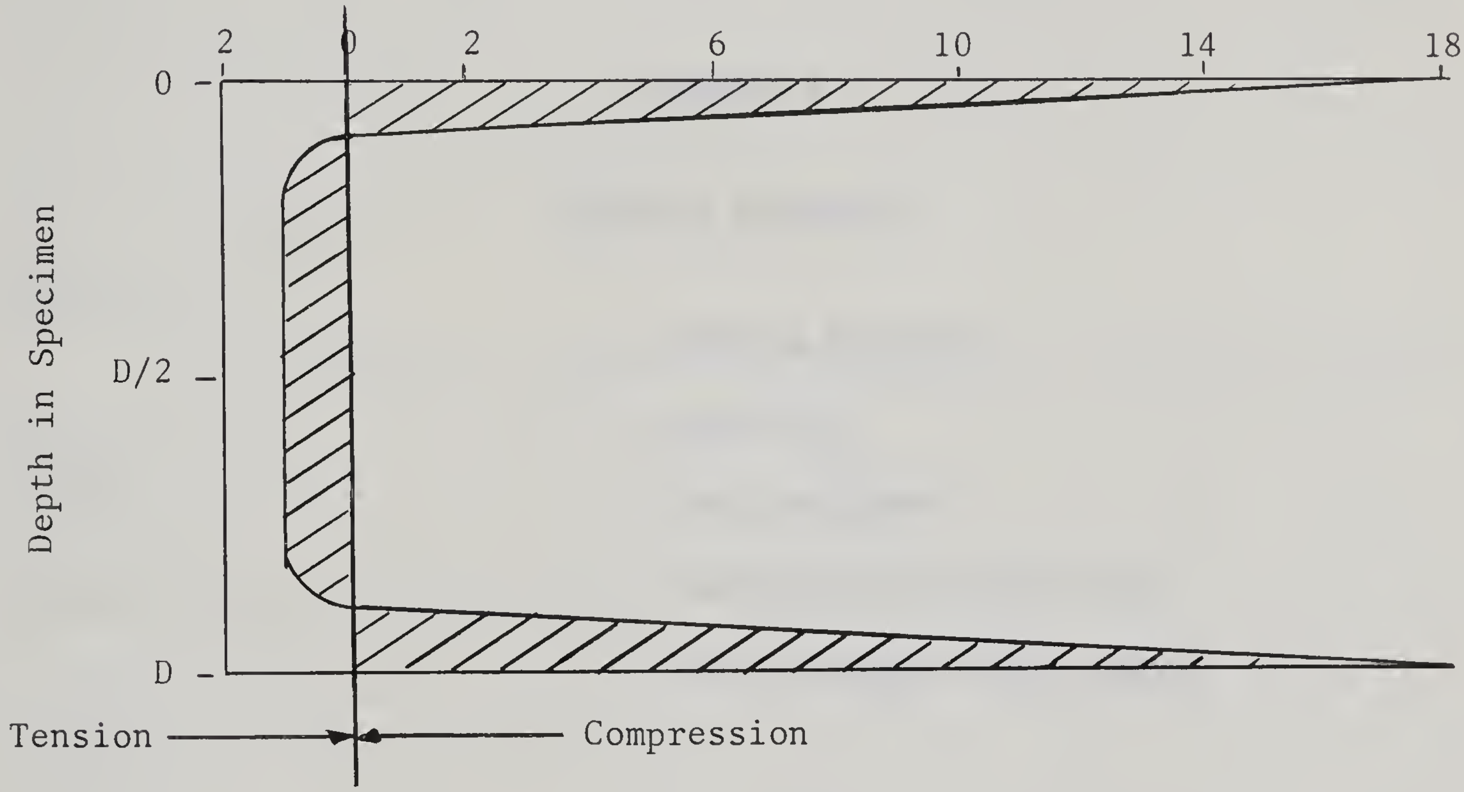

FIGURE A5. HORIZONTAL STRESS DISTRIBUTION ON Y-AXIS

APPENDIX B

TESTING APPARATUS

- Testing Machine
- Load Cell
- Load Indicator
- Calibration of Load Cell
- The Tuckerman Strain Gauge

Testing Machine

A 5 ton capacity Wykeham Farrance, Model 57, compression machine was used for applying load to the samples. The gear box of this machine has 6 - 5 to 1 reductions and additional gear wheels giving a total of thirty different rates of speeds ranging from .075 in/min to .000045 in/min. The upper cross frame of the machine is mounted to the main body by means of two spherical surfaced steel rods having a clearance distance of 13 3/4 inches. The upper ends of these rods are threaded to facilitate the positioning of the cross frame with respect to the position of the loading platform of the machine.

Load Cell

A 5 ton capacity, commercially manufactured, (Kyawa Electronic Instruments Co., Ltd.) load cell was employed as the load transducer. The load cell is composed of four strain gauge elements which combine to form a Wheatstone bridge circuit. With no load each strain element has a certain length and a resistance of 120 ohms. When the load cell is subjected to load the strain gauges shorten which cause a change in their resistances which in turn results in an unbalanced voltage across the bridge circuit. The magnitude of the unbalanced voltage is a measure of the load being applied to the load cell.

The power supply was obtained from a battery operated self-balancing load indicator which was also used as the load measurement device.

Load Indicator

Load measurements were recorded by means of a portable Kyawa Self-Balancing Load Indicator, Model SLW. The measurement values are indicated on a 240° wide-angle scale of 230 mm in length by a pointer deflection. The controls on the indicator consist of a zero adjuster, a span adjuster, a span control push-button and a power switch.

The operation of the load indicator is briefly outlined in the following paragraphs.

(1) The cable from the transducer is connected to a five-pin socket of the indicator and the power switch is turned on.

(2) The pointer on the scale is then adjusted to the zero position on the scale by means of the zero adjuster.

(3) The span control push-button is then pushed and the pointer should deflect to a position on the scale which corresponds to a calibrated scale value, supplied with the indicator. The button is then released and the pointer returns to a zero reading position.

(4) If the deflected reading does not correspond to the calibration scale value the span adjuster is turned until the "push-button" reading equals the calibrated scale reading.

(5) As load is applied to the load cell the resulting unbalanced voltage from the Wheatstone bridge circuit is reflected in the deflection of the pointer.

Calibration of Load Cell

The load cell was calibrated by employing a 200,000 lb capacity Baldwin-Tate-Emery Universal Testing Machine. This machine consists of two basic elements, a loading system and an indicating system.

The loading unit contains a fixed frame and a moving frame. The fixed frame consists of four columns rising vertically from a base, a top platen and a hydraulic cylinder bolted to the top of the top platen. The moving frame consists of an adjustable mounted crosshead mounted between the base and the top platen. The moving frame is supported by the fixed frame through hydraulic fluid in the cylinder.

The load cell was placed between the adjustable crosshead and the top platen of the machine. The self-balancing indicator was connected to the load cell and adjusted to zero by the method previously discussed. Load was applied to the load cell by the upward movement of the adjustable crosshead. Indicator readings were taken at every 500 lb increments of load (up to 10,000 lbs) as shown by the indicating system of the testing machine. This procedure was repeated three times and the indicator readings for a given load were averaged. FIGURE B1 shows the resulting load cell calibration curve.

The Tuckerman Strain Gauge

A complete description of the Tuckerman Strain Gauge is contained in the Tuckerman Optical Strain Gauge System, Instruction Manual No. 750, (1958). A detailed summary of the description and operation of the instrument was presented by Gillespie (1966) and for this reason only a

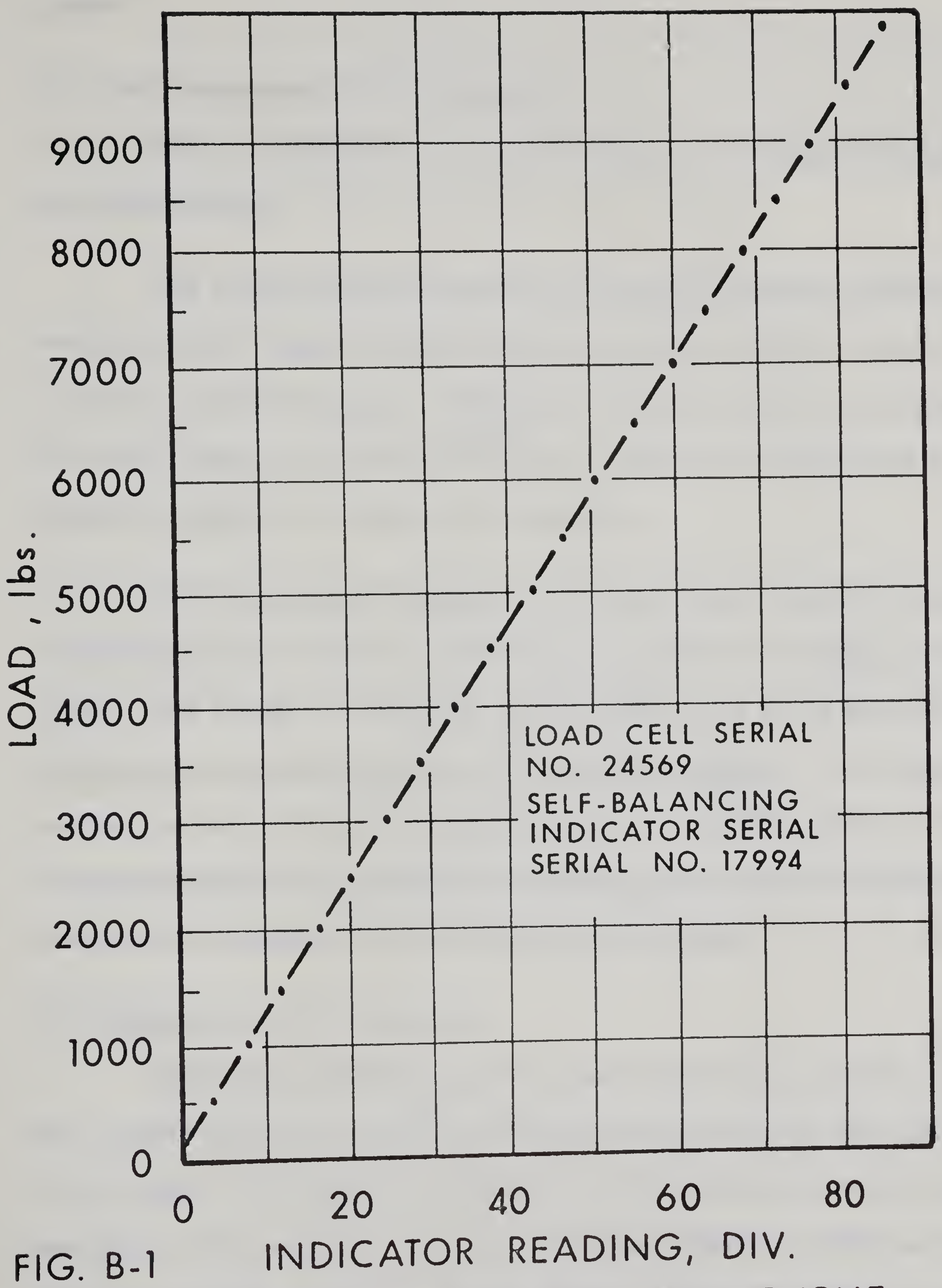

FIG. B-1
LOAD CELL CALIBRATION CURVE

brief description of the instrument is given in the following paragraphs.

(1) The Components of the Instrument

The main components of the instrument are the autocollimator and strain gauge.

The autocollimator consists of a highly corrected objective lens having a focal length of approximately 250 mm, a reticule having a uniformly graduated scale, a fiduciary line and vernier positioned in the focal plane of the objective lens, a lamp for illuminating the fiduciary line and vernier and an eyepiece.

The strain gauge consists of a steel body to which is attached a knife edge and a mirror system which consists of a lozenge and a roof prism. The lozenge is attached to the gauge so as to be able to rotate about an axis perpendicular to the axis of the gauge. The outside surface of one side of the lozenge serves as a movable mirror. The roof prism forms two perpendicular mirrors which can be rotated by means of an adjustment screw attached to the gauge.

(2) Operation of the Instrument

Light from the fiduciary line and vernier scale passes through the objective lens of the autocollimator and falls upon the roof prism of the gauge. The light is reflected by the roof prism to the lozenge and back to the objective lens of the autocollimator to form an image on the scale of the reticule. As the surface to which the gauge is attached deforms, the lozenge rotates and the image on the reticule moves accordingly.

(3) Determination of Strain

The reticule scale consists of 125 graduations, 25 of which are numbered. Each numbered division represents 0.001 radian rotation of the lozenge and therefore the entire scale of 25 numbered divisions correspond to 0.025 radian rotation of the lozenge.

The relation between the 25 numbered divisions and strain can be calculated from the equation

$$\text{Strain} = \frac{LR}{1000E}$$

where L represents lozenge size, R represents total number of transversed divisions and E the gauge length. Calibration factors for the auto-collimator and lozenge are introduced into the above equation so that the actual strain reading in inches per inch is given by the equation

$$\text{Strain (inches per inch)} = \frac{LRFA}{1000E}$$

where L, R and E are as before, F and A are the calibration factors for the lozenge and autocollimator respectively, both of which are factory supplied.

APPENDIX C

DETAIL TESTING PROCEDURE
and
EXAMPLE CALCULATIONS

- Calculation of Tensile Stresses
- Strain Prior to Failure
- Indirect Method of Determining Failure Strain
- Stiffness Modulus at Failure
- Rate of Deformation

Detail Testing Procedure

The following section is devoted to a detailed description of the testing procedure used in determining the low temperature tensile properties of the Marshall specimens and asphalt concrete cores.

(1) The unit weight of each sample was determined according to ASTM method of test D1188. The average thickness of the samples was determined to the nearest 0.01 inches by means of calipers. All samples were 4 inches in diameter.

(2) Each sample was closely inspected. Many of the samples contained surface irregularities, such as large air voids, and since the samples were subjected to a compressive loading condition the samples were orientated such that the flaws were outside the area of contact with the loading strips. The horizontal and vertical diameters of the samples were marked for correct positioning in the testing machine.

(3) Aluminum blocks, 1/4" x 1/4" x 1/8" with scribed lines were fixed to the samples by means of asphalt cement. These blocks were placed one inch apart along the horizontal axis on either side of the vertical axis of the samples. A tool with a gauge length of one inch between knife edges was placed in the scribed gauge marks to ensure a proper gauge length.

(4) The samples were placed in a temperature controlled room for a period of 24 hours prior to testing. The temperature in the room was kept at $0^{\circ}F$ ($\pm 3^{\circ}F$) throughout the testing procedure.

(5) Prior to testing, the Tuckerman Strain Gauge was positioned on the sample and held in place by an elastic band attached to the gauge and encircling the sample. The knife edges of the strain gauge were firmly seated in the scribed lines of the gauge marks.

(6) The sample was positioned in the testing machine with plywood loading strips, 3" x 1/2" x 1/4", placed at the top and bottom of the vertical axis of the samples. The loading platform of the machine was raised slightly so as to firmly seat the sample and loading strips between the steel plate on top of the load cell and the upper cross frame of testing machine. The self-balancing load indicator reading was adjusted to zero and the dial gauge that measures the vertical movement of the loading platform of the compression machine was magnetically attached to the upper cross frame of the machine with the stem of the dial resting on the loading platform of the machine.

(7) The strain gauge was adjusted in order to obtain a tension image in the eyepiece of the autocollimator. If the reflected light from the lozenge formed an image on the scale of the autocollimator the lozenge was held in place while the knife edge of the strain gauge was moved in a direction that simulates a tensile stress. If the reflected image moved up the scale, a compression image, the roof prism was adjusted, by means of the adjustment screw, until movement of the knife edge resulted in a downward scale movement, a tension image. The location of the tension image was then adjusted, by means of the roof prism adjustment screw, to the upper end of the scale to ensure a

maximum range of strain measurements during each test.

(8) The compression machine was set at a rate of loading of 0.06 in/min. A reading of the reflected image and the dial gauge measuring the vertical movement of the loading platform was taken at zero load and the test was started. The time from the beginning of a test to the passage of each 5 divisions on the load indicator was recorded in a room adjacent to the temperature controlled room. At each of these 5 divisions the strain gauge and strain dial were also read and these readings were relayed to the adjacent room, by means of an intercom system, where the readings were recorded.

(9) As a sample approached failure condition there was a rapid increase in the rate of movement of the reflected image across the scale of the autocollimator. At this time during each test the strain gauge and the strain dial were removed from their positions because of the damage which may have had occurred to them due to the sudden and brittle failure mode of the samples. Load indicator and time readings were recorded to failure of the samples.

(10) After failure the samples were examined for such characteristics as the occurrence of large voids and the amount of aggregate fracture along the failure planes of the samples. Such characteristics aided in explaining any anomalies in the test results.

Example Calculations

The following example calculations are based on the tests performed on the asphalt concrete cores extracted from the lower course of highway 57-A. The laboratory data sheets pertaining to these cores are presented at the end of this Appendix. Sample No. 2-B is used to show the calculations involved.

Calculation of Tensile Stresses

The tensile splitting stress is obtained by the equation $S_x = 2P/\pi dt$.

where: P = total load in pounds

d = diameter of sample in inches

t = thickness of sample in inches.

The value of P is obtained from the load cell calibration curve, FIGURE B1, for any load indicator reading. All samples were 4 inches in diameter. The tensile stress corresponding to any load indicator reading is calculated by dividing the value of $2P/\pi dt$ for that reading by the thickness of the sample.

Example: Maximum Tensile Stress, Sample No. 2-B.

Maximum indicator reading = 62

Maximum load = 7300 lbs. (FIGURE B1)

Diameter of core = 4.0 in.

Thickness of core = 2.70 in.

Maximum tensile stress $= \dfrac{2 \times 7300}{\pi \times 4 \times 2.7} = 430$ psi

Strain Prior to Failure

The total horizontal strain is calculated by the equation

$$\text{Strain (inches per inch)} = \frac{FALR}{1000E}$$

F (lozenge calibration factor) = 1.003

A (collimator calibration factor) = 1.003

L (lozenge size in inches) = 0.2

R (transversed divisions)

E (gauge length) = 1.0

The only variable in this formula is R and therefore

$$S = \frac{1.003 \times 1.003 \times 0.2 \times R}{1 \times 1000} = 2.012 \times 10^{-4} \text{ inches/in./division}$$

The assumption was made that Poisson's ratio was equal to 0.33 and the tensile strain caused by the tensile stress was equal to half the measured strain (by the law of superposition).

Tensile strain = 1.006×10^{-4} inches/in./division

Example: Sample No. 2-B

Tensile Strain Prior to Failure;

Initial fiduciary reading = 17.7

Final fiduciary reading = 11.0

Divisions transversed = 6.7

Tensile strain = $1 \times 10^{-4} \times 6.7$

$= 67 \times 10^{-5}$ inches/in.

The tensile stresses and tensile strains were calculated for every five divisions transversed by the load indicator pointer and a tensile stress-strain curve was plotted for each sample. These curves did not include failure conditions because the strain gauge was removed from the samples prior to failure and hence tensile strain at failure could not be directly determined from experimental results.

Indirect Method of Determining Failure Strain

A relationship was found to exist between the time elapsed from the beginning of a test and the plastic tensile strain developed in a sample prior to failure. Using this relationship an approximate tensile failure strain was determined for each sample. This relationship is best explained by an example.

From FIGURE C1 the tensile strain (elastic strain) at the proportional limit of sample no. 2-B equals 24 x 10^{-5} in/in. The total tensile strains, as determined by the last three readings of the strain gauge and the time elapsed from the beginning of the test to these readings are:

Time (mins-secs)		Tensile Strain (in/in x 10^{-5})
3	56	38
4	06	51
4	15	67

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[illegible]

[illegible]

[illegible]	[illegible]

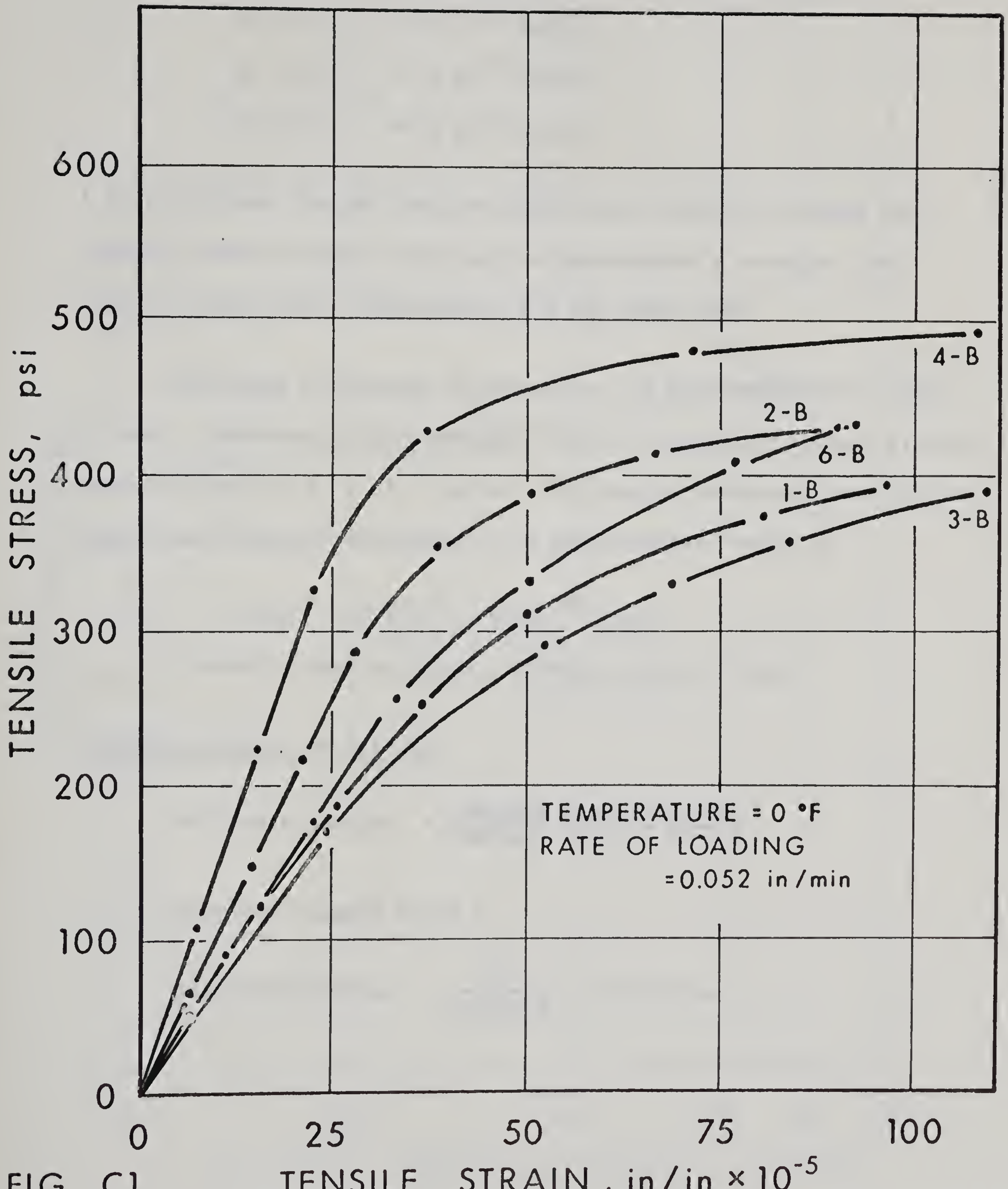

FIG. C1

TENSILE STRESS-STRAIN RELATIONSHIPS, HIGHWAY 57-A

The plastic tensile strains corresponding to these values are:

$$38 - 24 = 14 \times 10^{-5} \text{ in/in.}$$

$$51 - 24 = 27 \times 10^{-5} \text{ in/in.}$$

$$67 - 24 = 43 \times 10^{-5} \text{ in/in.}$$

A plot of times elapsed from the beginning of the test versus log plastic tensile strains was found to approximate a straight line. FIGURE C2 shows this relationship for the above data.

The time to failure of sample no. 2-B was recorded as 4 mins. 22 secs. Extension of the straight line to this time yields a plastic tensile strain of 67×10^{-5} in/in. The tensile failure strain of the sample was therefore estimated to be approximately equal to:

$$24\text{x}10^{-5} + 67\text{x}10^{-5} = 91\text{x}10^{-5} \text{ in/in.}$$

(elastic tensile strain + plastic tensile strain)

Stiffness Modulus at Failure

$$\text{Stiffness Modulus} = \frac{\text{Maximum Tensile Stress}}{\text{Maximum Tensile Strain}}$$

Example: Sample No. 2-B.

$$\text{Stiffness Modulus} = \frac{430}{91\text{x}10^{-5}} = 472{,}000 \text{ psi.}$$

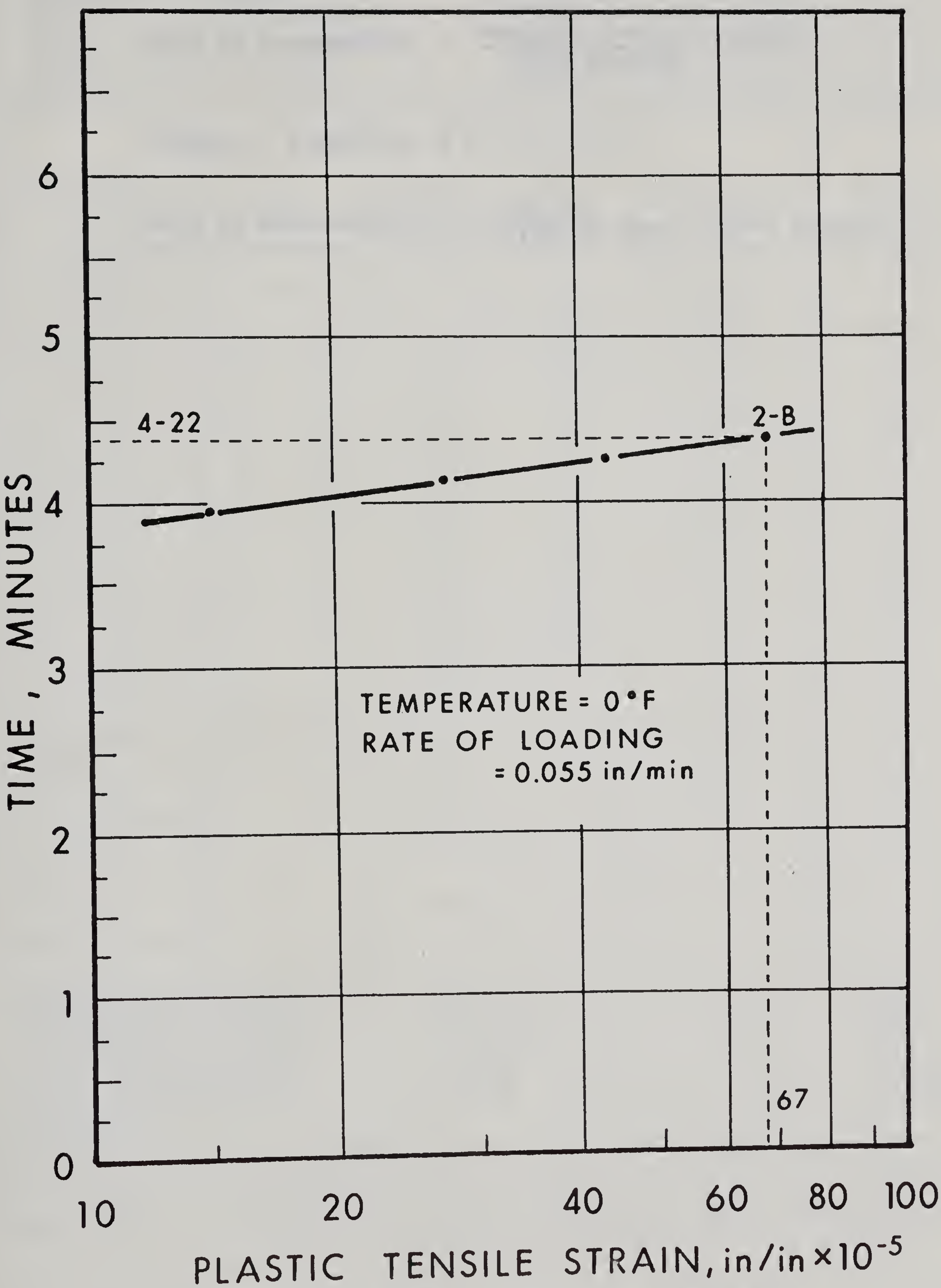

FIG. C 2. TIME - PLASTIC TENSILE STRAIN RELATIONSHIP

Rate of Deformation

$$\text{Rate of Deformation} = \frac{\text{Vertical Strain Dial Rd}}{\text{Time Elapsed}}$$

Example: Sample No. 2-B.

$$\text{Rate of Deformation} = \frac{.198\ \text{in.}}{3\ \text{mins}\ 38\ \text{secs}} = .0545\ \text{in/min.}$$

UNIVERSITY OF ALBERTA DEPT. OF CIVIL ENGINEERING HIGHWAY MATERIALS LABORATORY TENSILE SPLITTING TEST	
	LOCATION Hwy. 57-A
	SAMPLE No. 2-B DIAMETER 4 ins.
	TEMP. 0 °F THICKNESS 2.70 ins.
	DENSITY 143.0 lb/ft^3 DATE 15/9/66

Load Indc. Rd. (Divisions)	Time (mins-secs)		Tensile Stress (psi)	Vertical Strain Dial Rd. (ins)	Fiduciary Rd. (Divisions)	Tensile Strain ($in/inx10^{-5}$)
0				.100	17.7	
5	0	10	31	.109	17.3	4
10	1	18	71	.175	17.0	7
15	2	15	108	.232	16.6	11
20	2	45	147	.250	16.3	14
25	3	03	181	.266	16.0	17
30	3	20	216	.280	15.6	21
35	3	28	252	.290	15.3	24
40	3	38	284	.298	14.9	28
45	3	47	318		14.5	32
50	3	56	353		13.9	38
55	4	06	386		12.6	51
60	4	15	417		11.0	67
62	4	22	430			91

MODULUS OF ELASTICITY	1,050,000	psi
STIFFNESS AT FAILURE	472,000	psi
RATE OF DEFORMATION	.0545	in/min
REMARKS:		

UNIVERSITY OF ALBERTA DEPT. OF CIVIL ENGINEERING HIGHWAY MATERIALS LABORATORY TENSILE SPLITTING TEST	LOCATION Hwy. 57-A SAMPLE No. 1-B DIAMETER 4 ins. TEMP. 0 °F THICKNESS 1.57 ins. DENSITY 142.5 lb/ft^3 DATE 14/9/66

Load Indc. Rd. (Divisions)	Time (mins-secs)		Tensile Stress (psi)	Vertical Strain Dial Rd. (ins)	Fiduciary Rd. (Divisions)	Tensile Strain ($in/in x 10^{-5}$)
0				.000	18.6	
5	0	55	54	.043	17.9	7
10	2	51	122	.150	17.0	16
15	3	47	186	.190	16.1	25
20	4	14	253	.210	15.0	36
25	4	34	311	.222	13.6	50
30	4	53	372		10.5	81
31	5	00	395			96

MODULUS OF ELASTICITY	760,000	psi
STIFFNESS AT FAILURE	412,000	psi
RATE OF DEFORMATION	.0500	in/min
REMARKS:		

UNIVERSITY OF ALBERTA DEPT. OF CIVIL ENGINEERING HIGHWAY MATERIALS LABORATORY TENSILE SPLITTING TEST	LOCATION Hwy. 57-A SAMPLE No. 3-B DIAMETER 4 ins. TEMP. 0 °F THICKNESS 2.32 ins. DENSITY 141.6 lb/ft^3 DATE 13/9/66

Load Indc. Rd. (Divisions)	Time (mins-secs)		Tensile Stress (psi)	Vertical Strain Dial Rd. (ins)	Fiduciary Rd. (Divisions)	Tensile Strain ($in/in x10^{-5}$)
0				.100	19.9	
5	0	30	37	.129	19.6	3
10	1	53	83	.202	19.0	9
15	3	12	126	.271	18.2	17
20	3	45	171	.296	17.5	24
25	4	04	210	.313	16.9	30
30	4	20	252	.325	16.7	32
35	4	33	293		14.7	52
40	4	45	331		13.0	69
44	4	57	362		11.5	84
47	5	15	387			113

MODULUS OF ELASTICITY	1,230,000	psi
STIFFNESS AT FAILURE	342,000	psi
RATE OF DEFORMATION	.0525	in/min
REMARKS:		

UNIVERSITY OF ALBERTA DEPT. OF CIVIL ENGINEERING HIGHWAY MATERIALS LABORATORY TENSILE SPLITTING TEST	LOCATION Hwy. 57-A SAMPLE No. 4-B DIAMETER 4 ins. TEMP. 0 °F THICKNESS 1.79 ins. DENSITY 142.0 lb/ft^3 DATE 13/9/66

Load Indc. Rd. (Divisions)	Time (mins-secs)		Tensile Stress (psi)	Vertical Strain Dial Rd. (ins)	Fiduciary Rd. (Divisions)	Tensile Strain (in/inx10^{-5})
0				.100	18.7	
5	0	35	47	.130	18.4	3
10	2	10	107	.222	17.9	8
15	3	17	164	.274	17.5	12
20	3	49	222	.296	17.2	15
25	4	09	272	.311	16.9	18
30	4	25	327	.322	16.5	22
35	4	38	380		16.0	27
40	4	51	430		15.0	37
45	5	05	480		11.5	72
46	5	12	490			108

MODULUS OF ELASTICITY	1,510,000	psi
STIFFNESS AT FAILURE	453,000	psi
RATE OF DEFORMATION	.0510	in/min

REMARKS:

UNIVERSITY OF ALBERTA DEPT. OF CIVIL ENGINEERING HIGHWAY MATERIALS LABORATORY TENSILE SPLITTING TEST	LOCATION Hwy. 57-A SAMPLE No. 6-B DIAMETER 4 ins. TEMP. 0 °F THICKNESS 2.32 ins. DENSITY 145.8 lb/ft^3 DATE 13/9/66

Load Indc. Rd. (Divisions)	Time (mins-secs)		Tensile Stress (psi)	Vertical Strain Dial Rd. (ins)	Fiduciary Rd. (Divisions)	Tensile Strain (in/inx10^{-5})
0				.100	18.2	
5	0	28	37	.128	17.9	3
10	1	44	83	.200	17.1	11
15	3	00	126	.276	16.4	18
20	3	29	171	.302	15.9	23
25	3	46	210	.318	15.3	29
30	4	00	252		14.9	33
35	4	13	293		14.0	42
40	4	25	331		13.2	50
45	4	35	362		12.4	58
50	4	47	412		10.5	77
52	4	57	427			93

MODULUS OF ELASTICITY	1,230,000	psi
STIFFNESS AT FAILURE	460,000	psi
RATE OF DEFORMATION	.0580	in/min
REMARKS:		

APPENDIX D

TEST RESULTS
and
PROPERTIES OF MARSHALL MIXTURE DESIGN

TABLE D-1

TEST RESULTS OF HIGHWAY 21-A-1

Core Number	Time to Failure (mins-secs)	Thickness (ins)	Density (pcf)	P.L. Stress (psi)	Maximum Stress (psi)	Elastic Strain (in/inx10^{-5})	Failure Strain (in/inx10^{-5})	Rate of Loading (in/min)	Stiffness (psix10^3)
Sec. A									
A-1	5 15	1.90	146.6	308	415	16	63	.054	658
A-2	4 42	1.72	146.9	284	515	15	108	.054	477
A-3	4 40	1.82	142.2	268	404	21	33	.059	1240
A-4	3 39	1.52	143.5	260	260	15	15	.053	1730
A-5	4 40	1.63	142.5	299	450	15	68	.054	660
Sec. B									
B-1	4 00	2.02	144.5	290	410	14	114	.052	360
B-2	5 36	2.22	147.2	263	492	21	107	.053	458
B-3	5 10	1.94	146.3	302	502	24	92	.053	545
B-4	4 00	2.11	145.7	363	435	22	64	.055	680
B-5	2 28	2.30	144.2	334	440	21	36	.058	1200
Sec. C									
C-1	4 13	1.98	143.8	400	454	20	32	.054	1420
C-3	4 45	1.76	142.5	332	415	32	67	-	600
C-4	5 50	1.47	142.5	270	385	12	25	-	1540
C-5	4 13	1.27	141.9	312	344	17	51	.055	690

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TABLE D-2

TEST RESULTS OF HIGHWAY 34-A-1

Core Number	Time to Failure (mins-secs)	Thickness (ins)	Density (pcf)	P.L. Stress (psi)	Maximum Stress (psi)	Elastic Strain (in/inx10^{-5})	Failure Strain (in/inx10^{-5})	Rate of Loading (in/min)	Stiffness (psix10^3)
Sec. G-1									
1a	5 05	1.96	146.9	391	548	17	53	.050	1040
2a	5 39	2.01	146.5	382	607	17	75	.058	810
1b	4 15	2.12	142.3	91	362	7	107	.054	346
2b	5 00	2.06	148.3	330	540	26	156	.056	338
Sec. G-2									
4a	5 05	1.88	144.7	408	556	24	41	.055	1360
5a	5 25	1.94	144.9	444	524	24	39	.057	1340
4b	5 15	1.98	145.0	43	450	3	101	.055	445
5b	4 57	1.48	142.9	329	410	31	69	.057	595
Sec. G-3									
5a	5 20	2.28	143.3	377	495	17	77	.056	585
1a	5 00	1.88	142.5	259	500	17	97	.056	515
5b	5 10	2.25	145.6	302	525	28	76	.059	692
1b	5 02	2.31	144.7	294	503	27	82	.056	614
Sec. G-4									
1a	4 25	1.77	144.8	330	432	25	76	-	570
5a	4 38	2.28	144.7	214	392	20	110	.057	357
1b	4 42	2.02	141.4	42	354	8	155	.060	229
5b	4 55	2.16	141.8	39	406	4	115	.057	355
Sec. G-5									
2a	5 07	2.04	144.2	287	430	24	104	.054	414
4a	5 09	1.75	142.6	226	356	17	87	.052	410
2b	5 00	1.70	145.4	234	453	21	87	.052	520
4b	4 48	2.00	142.1	199	415	21	116	.057	358

TABLE D-3

TEST RESULTS OF HIGHWAY 57-A

Core Number	Time to Failure (mins-secs)	Thickness (ins)	Density (pcf)	P.L. Stress (psi)	Maximum Stress (psi)	Elastic Strain (in/inx10^{-5})	Failure Strain (in/inx10^{-5})	Rate of Loading (in/min)	Stiffness (psix10^3)
Sec. L-1									
1A-A	4 57	2.37	142.8	206	372	12	92	.058	405
3A-A	4 45	2.50	143.3	195	368	20	70	-	525
4A-A	5 16	2.38	142.8	361	497	21	76	.053	625
6A-A	5 10	2.15	142.2	272	415	18	62	.054	670
1A-B	4 10	1.65	141.0	177	378	9	77	.054	490
2A-B	4 35	1.48	136.1	58	254	7	102	.054	250
3A-B	4 02	1.94	138.0	151	262	12	80	.052	328
5A-B	4 43	1.68	137.6	236	290	20	41	.056	710
Sec. L-2									
1A	4 45	2.14	141.2	39	318	9	159	.054	200
2A	5 00	2.23	142.5	38	453	4	94	.054	480
4A	4 47	2.12	143.8	90	405	13	107	.056	379
5A	4 55	2.00	141.1	42	348	5	95	.054	367
6A	5 17	2.41	142.5	35	452	4	99	.051	456
1B	5 00	1.57	142.5	122	395	16	96	.050	412
2B	4 22	2.70	143.0	252	430	24	91	.055	472
3B	5 15	2.32	141.6	37	387	3	113	.053	342
4B	5 12	1.79	142.0	272	490	18	108	.051	453
6B	4 57	2.32	145.8	37	427	3	93	.058	460

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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE D-4

TEST RESULTS OF MARSHALL SPECIMENS

Specimen Number	Time to Failure (mins-secs)	Thickness (ins)	Density (pcf)	P.L. Stress (psi)	Maximum Stress (psi)	Elastic Strain (in/inx10^{-5})	Failure Strain (in/inx10^{-5})	Rate of Loading (in/min)	Stiffness (psix10^3)
Supplier #1									
1	5 15	2.50	144.5	344	524	16	29	.055	1800
2	5 25	2.50	144.5	418	586	27	48	.052	1220
3	5 33	2.50	144.5	418	586	18	38	.051	1540
4	5 50	2.50	144.5	382	534	30	49	.053	1090
5	6 05	2.50	144.5	418	569	21	37	.051	1535
Supplier #3									
6	5 32	2.50	145.3	418	586	36	61	.056	960
7	5 06	2.50	145.3	234	580	13	57	.056	1020
8	5 40	2.50	145.3	382	555	45	85	.057	655
9	5 29	2.50	145.3	418	473	41	52	.054	910
10	5 48	2.50	145.3	308	562	16	53	.052	1060
Supplier #2									
11	5 15	2.50	144.6	308	489	32	75	.055	654
12	5 50	2.50	144.6	344	622	18	56	-	1110
13	5 50	2.50	144.6	234	502	29	105	.055	480
14	5 45	2.50	144.6	450	545	34	50	.055	1090
15	5 35	2.50	144.6	382	533	27	47	.050	1130

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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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TABLE D-5

PROPERTIES OF MARSHALL MIXTURE DESIGN

Supplier No.	Penetration Grade	Stability lbs	Flow .01 in	AC %	Density pcf	Voids %	VMA %	VF %
1	200/300	1330	8.3	5.4	144.5	3.9	14.1	72.5
2	200/300	1555	8.7	5.4	144.6	4.2	14.1	70.5
3	200/300	1645	10.4	5.5	145.3	4.1	13.9	70.0
Aggregate Gradation		% Passing		3/4"	#4	#10	#40	#200
Supplier Nos. 1, 2 and 3				100	51	34	15	6.7